JOURNAL

of the

AMERICAN WELDING SOCIETY

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SECTION OFFICERS AND HEADQUARTERS

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933 Bay St., Toronto

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AMERICAN WELDING SOCIETY
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PROGRAM

ANNUAL MEETING

AMERICAN WELDING SOCIETY

33 West 39th Street

April 23, 24 and 25, 1930

All Sessions Start Promptly as Scheduled

IMPORTANT NOTICE

Members and their guests are requested to register

WEDNESDAY, APRIL 23, 1930

Morning

Business and Technical Sessions

10:00 a. m. to 10:30 a. m., Room 2, 5th Floor, Business Meeting, F. T. Llewellyn, President, presiding.

Report of President, Report of Tellers' Committee, Election of Officers.

10:30 a. m. to 12 noon, Room 2, 5th Foor, Technical Session, H. L. Whittemore, Vice-Director, American Bureau of Welding, Presiding.

"Distribution of Stresses in Fillet Welds," by L. C. Bibber, Bureau of Construction and Repair, Navy Department.

"An Investigation of Welded Connections Between Beams and Columns," by C. D. Jensen and E. H. Uhler, Assistant Professors of Civil Engineering, Lehigh University.

Afternoon

Technical Session

2:00 p. m. to 5 p. m., Room 2, 5th Foor, Technical Session, F. R. Low, Chairman, A. S. M. E. Boiler Code Committee, and Editor Emeritus of "Power," Presiding.

WEDNESDAY AFTERNOON (*Continued*)

"Construction and Operation of a New Type of Welded Lime Stone Kiln," by H. E. Rockefeller, Development Department, The Linde Air Products Company.

Discussion of Proposed Specifications for Fusion Welding of Drums or Shells of Power Boilers.

An outline of the history of the development of these Specifications will be presented by Mr. C. W. Obert, Honorary Secretary of the Boiler Code Committee, which will be followed by a discussion of the subject by prominent engineers.

Evening

Board of Directors' Meeting

7:30 p. m., Board of Directors' Meeting, Building Trades Employers Association, 2 Park Avenue, corner 33rd Street, New York.

Meeting of the Board of Directors of American Welding Society. Appointment of committee chairmen and other officers. Committee reports. Plans for the coming year.

THURSDAY, APRIL 24, 1930

Morning

Architectural Session

10:00 a. m. to 12 noon, Architectural Session, Room 2, 5th Floor, J. J. Crowe, Member Executive Committee, Presiding.

"What the Architect Wants to Know About Welding," by Russell Whitehead, Editor "Pencil Points."

"What the Architect Should Know About Pipe Welding," by Webster Tallmadge, President, Webster Tallmadge, Inc.

Discussion as to what the architect should know about Structural Steel Welding will be led by Prof. F. P. McKibben, consulting engineer, and Mr. H. H. Moss, engineer, The Linde Air Products Company.

Luncheon

12:30 p. m., Engineers' Club.

A luncheon for members of the Society has been arranged at the Engineers' Club. Tickets should be reserved in advance; number limited. Price $2.00

Afternoon

Technical Session

2:00 p. m. to 5 p. m., Room 2, 5th Floor, Technical Session, F. T. Llewellyn, President, Presiding.

"Constructing Panel Heating by Welding at British Embassy," by R. A. Wolff, Wolff & Munier, Inc.

THURSDAY AFTERNOON (*Continued*)

"Battledeck Welded Floor Construction," by Lee H. Miller, American Institute of Steel Construction.

"Stiffness of Welded Beam Connections," by C. H. Jennings and A. A. Jakkula, Research Department, Westinghouse Electric & Mfg. Company.

Evening

Annual Dinner

6:30 p. m., Annual Dinner, Building Trades Employers Association, 2 Park Avenue, corner 33rd Street, New York.

The Annual Dinner, as last year, will be arranged as a stag affair. Extraordinary entertainment has been provided by the Dinner Committee. A large number of unusual prizes will be given to holders of lucky numbers as well as souvenirs. There will be no speakers. Tickets should be secured from the Secretary at once. Price $6.00 per person.

FRIDAY, APRIL 25, 1930

Morning

Technical Session

10:00 a. m. to 12 noon, Room 2, 5th Floor, Technical Session, F. P. McKibben, Member, Executive Committee, Presiding.

"An All-Welded Barge," by E. H. Ewertz, Consulting Engineer.

"The Needs of the Metropolitan District for Instruction in Welding," by R. F. McKay, International Oxygen Company.

"Welding of Tube Turns," by W. P. Curley, Manager of New York District, Tube Turns, Incorporated.

Afternoon

Structural Steel Committee
and
American Bureau of Welding

1:45 p. m. to 3:00 p. m., Room 3, 5th Floor, Meeting of Structural Steel Welding Committee, J. H. Edwards, Chairman.

All members of the Society are cordially invited.

Considerable progress has been made by the committee, which will be partially demonstrated by Stereopticon slides and discussions, indicating in a general way the results obtained to date and methods of analysis to be employed. Progress report.

3:00 p. m. to 5:00 p. m., Room 3, 5th Floor, Annual Meeting of American Bureau of Welding, C. A. Adams, Presiding.

Report of progress made by various research committees of the Bureau. Plans for future investigational activities. Election of officers. Appointment of Executive Committee.

Annual Dinner—American Welding Society

Plans are now well under way for the annual dinner of the American Welding Society. The dinner will be held on Thursday evening, April 24, in conjunction with the annual spring meeting of the Society. Great mystery surrounds the plans for this year's dinner. The committee has been working night and day preparing for the event, which they promise will surpass the very successful gatherings of previous years. Those who have attended the last two dinners realize that this will be no small task, and if successful will result in an affair that should not be missed under any circumstances.

A good dinner, with plenty to eat, no speeches, and an entertainment both novel and refreshing will be the order of the evening. Souvenirs and door prizes will add to the pleasures of the evening. An evening of complete relaxation and enjoyment is in store for everyone attending. It will be the only social function of the spring meeting, and everyone will be there. A wonderful opportunity to meet and visit with the members and friends of the society. Plan to attend, reserve the date, April 24. A cordial invitation is extended to all to join and make this the best dinner the society has ever held.

OBITUARY

Peter Junkersfeld

Peter Junkersfeld, vice-president of the Stone & Webster Engineering Corporation, died suddenly from a heart attack March 18 at his home in Scarsdale, N. Y. He was 60 years old and had been in his office on the same day.

Mr. Junkersfeld had a distinguished career in engineering and construction work. Soon after his graduation from the University of Illinois in 1895, Mr. Junkersfeld entered the service of the Chicago Edison Company and remained there for nearly twenty-four years, becoming head of the engineering department. In 1916 he became president of the Association of the Edison Illuminating Companies. He was appointed a member of the board of directors of Industrial Preparedness for Illinois and associate member of the Naval Consulting Board by Secretary of the Navy Daniels.

Mr. Junkersfeld was chairman of a subcommittee of the American Standards Association Committee on Codes for Pressure Piping and in this capacity cooperated with the Committee of the A. W. S. on the same subject.

Arc-Welded Buildings Not Circus Stunts*

But a few weeks past wistful smiles, with a metaphorical tear or two, were brought to the faces of New York electrical engineers by the noble gesture of two business men. "We apologize," said the business men in effect, "for fear that the riveting of structural steel by the indefatigable artisans engaged in the erection of our new buildings may in the weeks to come possibly, or probably, annoy our neighbors." A billboard, a sheaf of letters, bore the sad news to the community. *Mais quel geste!*

Electric arc welding has been applied to all too few new structures. Conservatism, lack of trained workers, discord over suitable or best structural assemblies, and, above all, failure by many municipal inspection boards to recognize welding as an adequate tool, have all delayed deplorably the age of quieter steel erection.

The recent announcement that two coming office buildings for power companies, one thirteen, the other fourteen stories high, are both to be of electrically arc-welded construction is an encouraging interlude; but here again, in the case of one, the old fear appears to return, for the lower four floors are to be a combination of arc-welded and riveted fabrication. An electrical manufacturer is starting work in an Eastern city upon what, it is anticipated, will be ultimately the largest building there. It will, if all goes well, be arc-welded.

The riveters' racket may be unknown tomorrow, and apologies unnecessary, but the leaders of the electrical industry can assuredly speed the day by adopting arc-welding for their new structures, pointing the way for others to follow. Why not, for example, weld substation steelwork?

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*Reprinted from April 5, 1930, issue of *Electrical World*.

Southern California Office Building

In the new Southern California office building in Los Angeles, riveted connections will be used to take all vertical loads and welding will be used to take all seismic loads throughout. The total amount of welding in connection with this building will be approximately 50,000 ft. of fillet welds, in sizes from ⅜ in. to 1¼ in., as well as a number of butt welds. About eighteen tons of welding wire will be required. The steel for the new Edison building is being furnished and erected by the Consolidated Steel Co. of Los Angeles. This company is also doing the welding, using twenty-four welding operators and machines.

Technical Papers

In scheduling papers to be published in the JOURNAL of the American Welding Society the Meetings and Papers Committee aims to maintain a proper balance between the various welding processes. The success of this endeavor requires a supply of good papers pertaining to each of the processes for which the Society stands. The papers should be of technical rather than commercial interest, and usually they should describe work actually done.

The cooperation of every member of the Society, and especially that of manufacturers and users, is asked in supplying papers that reflect the use of all processes. These should be sent to Society headquarters as promptly and as frequently as possible.

New Members Secured in "Member for Member Membership Drive" December 15th to April 1st, 1930:

Class A

New Members	*Proposer*
E. L. Mathy	H. W. Saunders
Chas. A. Whittier	W. A. Slack

Class B

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W. Ahlstrom	J. M. Jardine
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E. A. Boyke	P. W. Fassler
E. P. Boyer	
T. F. Bright	
A. G. Bussmann	L. R. Gurley
G. J. Carroll	W. A. F. Millinger
E. H. Cathcart	P. P. Fugitt
A. S. Douglas	M. P. Bailey
S. W. Dunn	C. W. Hopp
E. D. Edmundson	
H. English	F. E. Rogers
A. E. Focke	J. A. Weiger
Chas. S. Freeman	G. W. Swan
J. M. Gwinn, Jr.	G. W. Swan
F. A. Green	Chas. Kandel
H. S. Hanna	G. H. Montfort
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S. W. Hayes	W. Spraragen
E. R. Hodson	F. E. Rogers
Fred Huggins	F. E. Rogers
Sarah R. Isbell	A. G. Odell
H. Klingelhofer	P. W. Fassler
F. T. Lacy	Paul Ross
Joseph Lex	P. Fassler
John Lund	F. E. Rogers
J. Massey	
J. F. Minnotte	G. H. Danforth
J. A. Murphy	F. E. Rogers
W. W. Mayhew	W. Spraragen
W. R. Ost	W. Spraragen
J. Oudhoff	F. E. Rogers
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W. D. Patterson	
A. L. Rath	F. E. Rogers
S. N. Roberts	P. Fassler
E. H. Right	P. P. Fugitt
Paul Ross	A. G. Odell
E. Rosenberg	W. Spraragen
Robt. C. Sahlin	F. M. Anderson
J. H. Saunders	F. E. Rogers
Martin Stirling	J. H. Wallis
J. J. Stunzi	P. Fassler
C. E. Swift	A. M. Candy
S. Tepper	
W. F. Waterbury	F. E. Rogers
G. H. Winkler	James Burke
A. M. Wood	J. Darby
I. L. Wise	J. M. Jardine

Class C

R. C. Anderson	H. N. Ewertz
C. J. Bean	Joseph Cave
T. S. Bell	B. W. Horsman
F. M. Bernard	W. W. Reddie
E. J. Berens	Paul Ross
L. H. Black	M. S. Hendricks
E. B. Brown	R. D. Layman
E. L. Briggs	T. F. Stoddard
L. I. Blennerhasset	H. S. Farnsworth
Gene Call	G. H. Montfort
L. R. Canfield	W. W. Reddie
J. Centofanti	W. L. Warner

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Angus Duncan	W. A. F. Millinger
H. A. Given, Jr.	
A. G. T. Godwin	W. Spraragen
E. S. Groves	W. Spraragen
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R. A. Gogarn	
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F. W. Gillespie	O. J. Bornemann
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D. C. Maloney	W. R. Coleman
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D. Richardson	H. S. Card
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Joseph Stevens	W. Kyme
J. G. Smith	E. Jeffery
R. E. Swearengin	K. Kimball

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J. A. Schreuder	K. Kimball
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H. B. White	J. G. Norris
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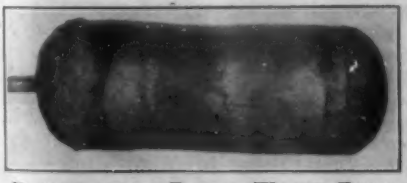
Karl Mayer	A. G. Odell
J. Martelon	A. G. Odell
A. T. Nortrud	Jud Hamman
John Pauls	A. G. Odell
S. J. Petras	A. G. Odell
John Petras	A. G. Odell
T. K. McManus	
L. Strousky	A. G. Odell
Chas. J. Walton	C. A. Britton
H. L. Webb	A. G. Odell

SECTION ACTIVITIES

Chicago

At the March 6th meeting of the Chicago Section, S. J. Mashl, of Van Schaack Bros. Chemical Works, Chicago, spoke on the subject of "Copper Welding." Following the reading of the paper and a short discussion, Mr. Mashl made up a test tank as follows: The shell was made of a nine-inch length of seamless copper tubing of four inches diameter and 12-gage wall thickness. The ends were dished ends of about the same thickness. In one of these ends a heavy piece of copper tubing had been welded from the inside, and the other end was threaded to fit the connection on a hydraulic testing pump. Mr. Mashl



DEMONSTRATION TEST OF WELDED TANK

set this up on an ordinary welding table and made a plain butt weld at each end. Then he filled the tank with water and applied pressure with a hydraulic pump up to 1000 pounds, which was the maximum capacity of the gage. There were no cracks, pin holes or other signs of distress in the welds. The photograph shows that there was considerably more bulging at the ends of the tank than in the center.

Detroit

The March meeting of the Detroit Section was held on the 31st. Burnside R. Value, executive engineer for Parsons, Klapp, Brinckerhoff & Douglas, consulting engineers for Detroit and Canada Tunnel, gave a very interesting illustrated lecture on the construction work and engineering features of the tunnel. Welding was used instead of calking the seams for the ⅜-inch tubes, about ten miles of ¼-inch bead being used. The thrust members for the hydraulic jacks used in driving the shield were welded to the tubes. Only one I-beam member being loosened so far. Gas torch cutting was used for evening up the steel piling driven for construction work. Mr. Value also stated that the shield from the Detroit side met the first tube that was sunk in the river so closely that they were welded directly together. An unexpected test of a welded member was made when one of the tubes rolled into shallow water at launching, bending the outside reinforcing frame but not breaking any welds. The lecture was well received by about forty guests and thirty-five members.

Northern New York

The Northern New York Section held a meeting on March 6 at the Hotel Ten Eyck in Albany attended by 68 members and guests. Dr. Frary of the Aluminum Company of America gave a lecture on aluminum. He outlined the compounds containing aluminum as they occur in nature and the methods of reducing these compounds to the pure metal. Numerous samples of aluminum bearing ores and byproducts of the reduction of these ores were shown. Methods of hardening aluminum and of obtaining different alloys were discussed. The

uses of aluminum are increasing rapidly. Besides being used for household utensils and furniture, the use of aluminum is rapidly increasing for building purposes, such as shingles and structural shapes. Following Dr. Frary's lecture, motion picture films, "The Electric Needle" and "The Light of a Race," were shown.

Philadelphia

Another exceptionally good meeting was held on March 17. The four-reel "talkie" on "Atomic Hydrogen Welding" went over big and everybody seemed much interested in it. There was quite a discussion after the showing of the picture.

On Monday evening, April 21, Prof. C. D. Jensen, of Lehigh University, and A. B. Kinzel, of Union Carbide & Carbon Research Laboratories, will talk on "Inspection of Welds."

Some of the Contributors to This Issue

Leon C. Bibber, author of the paper on "Distribution of Stresses in Fillet Welds," graduated from the University of Michigan in 1926 with the degree of Bachelor of Marine Engineering. He received his Master's Degree a year later from the same University. He has been connected in the designing and estimating departments of the Newport News Shipbuilding & Dry Dock Company, and from 1926 to 1929 was assistant in welding research at the Newport News Shipbuilding & Dry Dock Company. He is now Senior Welding Engineer of the Bureau of Construction and Repair, Navy Department.

Eric H. Ewertz, Consulting Engineer and author of the paper on "An All-Welded Barge," and inventor, is a graduate Mechanical Engineer and Naval Architectural Engineer. He has had practical experience as machinist, moulder and shipbuilder and has designed torpedoes at Schwartzkopen. He had charge of the construction of nearly two hundred vessels, naval and commercial, including ninety submarines and forty destroyers. Mr. Ewertz was also in charge of building Diesel engines, Weir pumps, powerplant equipment, oil refinery machinery, etc. He supervised all the welding activities for a large steel corporation and has assisted in the designing and construction of all-welded construction of vessels, buildings, special machinery and general construction work.

A. A. Jakkula, a joint author with C. H. Jennings of the paper "Stiffness of Welded Beam Connections," was born on March 19, 1904, and was graduated from the Civil Engineering Department of the University of Minnesota in 1926. A year later he received a Master's Degree from the same university. He has been connected with the Highway Department of Minnesota, U. S. Bureau of Public Roads, the University of Minnesota and the University of Michigan, as well as with the research department of the Westinghouse Electric & Manufacturing Company. At the present time he is instructor in Civil Engineering at the University of Michigan. He has written a number of papers, mostly on cements and concrete. He is a member of the American Society of Civil Engineers, and Tau Beta Pi, Sigma Xi, and Chi Epsilon.

Charles H. Jennings, joint author of the paper on "Stiffness of Welded Beam Connections," graduated from the Mechanical Engineering Department of Iowa State College in 1928 with a Degree of Bachelor of Science in Mechanical Engineering. He entered the Graduate Student Course of the Westinghouse Electric & Manufacturing Company in 1928, and in 1929 became connected with the Mechanics Section of the Research Department.

Cyril D. Jensen, Assistant Professor of Civil Engineering, and joint author with Prof. E. H. Uhler of the paper on "An Investigation of Welded Connec-

tions Between Beams and Columns," graduated from the Civil Engineering Department of the University of Minnesota in 1921. He has had four years of practical experience, including highway location and hydro-electric surveys. Since 1929 he has been teaching civil engineering at Lehigh University, and received his Master's Degree from that university in 1929.

E. H. Uhler, joint author with Prof. C. D. Jensen of the paper on "An Investigation of Welded Connections Between Beams and Columns," graduated from Lafayette College in 1928 with a Degree of Civil Engineering. He has had ten years of structural experience, including such organizations as the American Bridge Company and Pennsylvania Railroad. For the last twelve years he has been teaching civil engineering at Lehigh University.

Richard A. Wolff, President of Wolff & Munier, Inc., heating and ventilating and steam power engineers and contractors, is author of the paper on "Constructing Panel Heating by Welding at British Embassy." He graduated from Stevens Institute of Technology in 1914, is past president of the New York Chapter of the American Society of Heating and Ventilating Engineers, and member of the Standards Committee of the Heating and Piping Contractors, National Association, and is also a trustee and member of the Finance Committee of the Trustees of Stevens Tech., and vice-president of the New York Building Congress.

EMPLOYMENT SERVICE BULLETIN

Opportunities.—The Society is glad to learn of desirable opportunities from responsible sources, announcements of which will be published without charge in the BULLETIN.

Services Available.—Under this heading brief announcements (not more than seventy-five words in length) will be published without charge to members. Announcements will not be repeated except upon request received after an interval of three months; during this period, names and records will remain in the office reference files.

Note.—Copy for publication in the BULLETIN should reach the Society's Office not later than the thirtieth of the month if publication in the following issue is desired. *ALL REPLIES should be addressed to the number indicated in each case and mailed to Society headquarters.*

POSITIONS VACANT

V-50. Working Shop Foreman wanted for job welding shop located in New York employing ten men. Must be thoroughly experienced, all around man, especially on automotive welding and finishing; used to handling men. Either a machinist or welder by trade who can obtain maximum output at low cost and without comebacks. Moderate salary and bonus to start. Don't apply unless you have successfully handled a shop of this size or larger. Write full details in first letter in strictest confidence.

SERVICES AVAILABLE

A-94. Electric Welder desires position. Have had six years' experience installing, instructing and managing Welding Department.

A-95. Electric welder desires position. Wide experience in high pressure work. Good reference.

Panel Heating

An Application of Welding*

RICHARD A. WOLFF†

A SYSTEM of heating popular in England for twenty years was introduced to the United States of America through its use in the new British Embassy building at Washington, D. C. This system could not be used without the use of welding.

This system, which is fully patented, is similar to the ordinary low pressure hot water radiator system, with the exception that the heating surface in the various rooms consists of the surfaces of the ceilings

FIG. 1—THE BRITISH EMBASSY

E. L. Lutyens, R. A. of London, Architect. Frederick H. Brooke, Associate, Washington, D. C.

Richard Crittall & Co., Ltd., of London, Consulting Engineers, Jaros & Baum, Associates, New York City

themselves and in some cases of the walls or floors. These surfaces are warmed by jointless coils of piping embedded in the structure.

These coils take the place of radiators in the ordinary system, the hot water circulating through them either under a gravity or an accelerated head as may be found expedient.

The coils are placed in position at a suitable stage in the progress of the building, various methods being adopted to meet different types of building construction.

*Paper to be presented at Annual Meeting, A. W. S., April, 1930.

†Wolff & Munier, Incorporated.

The coils are connected to the mains and circulating branches by welded joints.

The warmth provided by the PANEL system is distributed almost entirely by radiation at comparatively low temperatures usually ranging from 95 deg. Fahr. to 130 deg. Fahr.

The human occupants of a PANEL heated room therefore receive their warmth direct from the panels and the reflecting surfaces of the walls, floors and furniture. Their feeling of comfort is thus largely independent of the temperature of the air and resembles very closely the feeling of exhilaration produced by mild sunshine in cool still air.

FIG. 2—WELDER AT WORK

In its original form, the PANEL system was intended to overcome the unsightliness and obstruction caused by exposed pipes and radiators. Its many advantages from other points of view have become apparent during the subsequent years in which it has been developed by the patentees.

Cost: As regards first cost, the price of the PANEL heating installation exceeds by little that of a first class radiator system for buildings of modern high class design and of medium to large size, provided the cost of enclosing radiators and pipes is taken into account as well as the saving due to the elimination of painting and decorating the exposed parts of the apparatus.

A peculiar advantage is that the system can be installed and completed at a very early stage in the construction of the building, which enables its use to dry out the building and eliminates the customary expense of providing temporary radiator connections.

The greatly extended use of concrete, both plain and reinforced, has produced the means of reducing very largely the initial cost of installa-

tion by the method of casting the panel pipes in the soffit of the flooring itself, thus obviating the necessity of additional plaster.

The methods of installation now adopted render the system applicable to any system of construction and involve no special provision in decorative schemes. The panels are almost exclusively incorporated in the ceiling, and the only stipulations necessary are that accommodation can be found for the panel pipes at the correct position in the soffits and that the plastering be carried out in accordance with the special instructions issued.

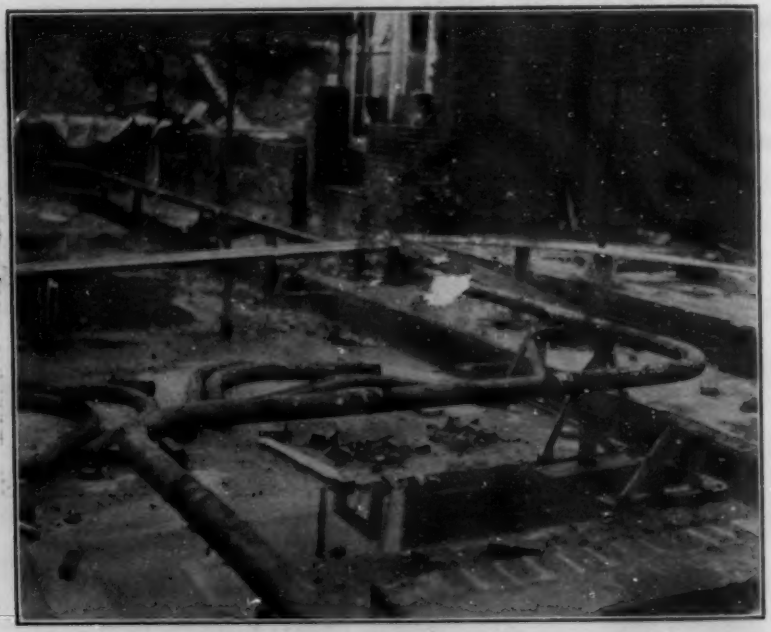


FIG. 3—PORTION OF THE RETURNED PIPING WELDED AND READY TO BE LOWERED INTO THE TRENCH

Standardized methods have been evolved for accommodating patent panel pipes in the soffits of all the recognized forms of concrete, hollow tile and wood joist floors, and the plastering can be carried out with the usual materials by any good plastering contractor in such a way as to insure absence of cracking.

Where wall and floor panels are necessary, they may be covered either by plaster, glass, marble, stone, mosaic tiles or similar material.

The importance of welding in this system is apparent as the coils are cast right into the concrete and are behind plaster and decorations, and no one would be interested in installing such a system if walls and ceilings had to be torn down to get at leaks. And would not pipe joints and fittings be too risky? In many cases quite aside from the risk of

leaks, fittings could not be used as it was necessary to make such close connections to get the pipes in the space allowed by the construction.

The coils installed in the British Embassy, over two hundred in number, were constructed of ½ in. extra heavy steel pipe. They were return bend coils made each of several pipe lengths and without a fitting or pipe joint. They were electrically welded, and before shipment were subjected to a test of 500 lb. air under water. Because of fear of an obstruction in the pipe or an inside weld, they were further tested by blowing a marble, slightly less in diameter than the inside diameter of the pipe, through the coil.

FIG. 4—COILS READY TO FORM AN INTEGRAL PART OF THE STRUCTURE

The connections to and from the coils were welded by the oxygen acetylene method. The return main ran all around the building in trenches and was similarly welded. The coils and piping, after installation and before being plastered over, were tested hydrostatically at a pressure of 200 lb. Substantially, the only piping in the PANEL heating system not welded was that in the boiler room where fittings were used. This represents such a small part of the total piping that the job can well be referred to as a welded job.

Illustration No. 1 shows the building itself.

Illustration No. 2 shows one of the welders at work.

Illustration No. 3 shows a portion of the return piping welded and ready to be lowered into the trench.

Illustration No. 4 shows some of the coils ready to form an integral part of the structure.

I desire to quote from a paper presented by Messrs. J. L. Musgrave and W. A. E. Taylor, of Richard Crittall & Co. Ltd., before the general meeting of the British Acetylene and Welding Association, on April 7, 1927.

"The reliability of the oxy-acetylene process if carefully carried out is proved by the results of the tests on three panel heated buildings, the contracts for which were carried out simultaneously.

"There were 4341 panel coils comprising 270,000 feet or 51 miles of pipe, 23,404 bends, with 15,850 electric welds. The approximate length of pipe used in addition to the panel coils was 125,800 feet, or about 24 miles with oxy-acetylene welds numbering about 13,000.

"So far not one leak has developed after the work has been cast in. Two of the systems have had two seasons' use; the third, owing to delay in completion of the building, has not yet been subjected to heat, but has stood filled with water for a long period."

I desire to take this opportunity to express my appreciation to the Linde Air Products Company. Welding on such a large and important scale was new to my firm, who installed this work in the new British Embassy under license of Richard Critall & Co., Ltd., the patentee of the system, and the aid we received from the Linde Air Products Company in training our men, and inspecting and supervising their work until they were pronounced proficient, gave us confidence in our undertaking, and the results obtained made us enthusiastic boosters of welding.

Testing Steel Sheet for Weldability

DE WITT ENDICOTT†

ONE of the fundamental considerations for successful oxy-acetylene welding of steel sheet is the careful selection of good material. Inferior sheet is bound to produce an unsatisfactory result. It does not matter how skilled the welder may be or how high grade his welding equipment is, if he is handicapped by poor grade material the finished product will not be up to standard.

Chemical composition alone is not sufficient indication of the quality of steel sheet. Welding qualities are determined to a great extent by the presence or absence of minute amounts of impurities in the steel. This holds especially true for thin sheet steel. Through lack of proper supervision in manufacture, steel sheet may at times contain a variety of non-metallic inclusions or impurities, most of them the product of the furnace work. These impurities are sometimes so small that it requires a good microscope to detect them. They do not show up in the chemical analysis, but they have a very decided effect on the behavior of the steel under the welding flame. It is always advisable, therefore to specify that the steel shall be suitable for welding and as a further check to test lots

†Technical Publicity Dept., Linde Air Products Co.

of steel as they are received from the mill. Such a test is easily made in the following manner:

Select a piece of sheet from the lot, about 6 in. square, and place it flat on the welding table. Fit a welding tip to the blowpipe one size smaller than that prescribed for work on your welding chart. Light the blowpipe and adjust the flame to neutral. Hold the blowpipe so that the tip of the inner cone of the flame is about ⅛ in. away from the sheet. Move the flame slowly along a straight line so that the sheet will be melted without burning clear through. After a strip about 3 in. long has been melted in this way, hold the blowpipe still until a hole is burned through the sheet.

If the sheet is high grade the path followed by the blowpipe will be free from an excess of oxide or scale and regular and smooth on the upper side. The under side of the test sheet will show a slight sag of smooth metal, free from oxide. The hole will be round and have smooth, rounded edges.

Sheet of inferior quality will show an oxide deposit on the upper side, part of which will be porous or flaky scale. There will also be a succession of irregular ripples, higher in the center than on the edges. The under side will show a sag that is covered with oxide. There may even be a series of small holes blown through by the sparking action of the impurities. In poor sheet the metal will be burned through very soon after the blowpipe is held stationary, in much less time than is necessary to put a hole in good sheet. The hole, incidentally, will be irregular with rather ragged edges. When the test sheet cools, examination will disclose porous, spongy looking globules of oxide on the edges of the hole.

This simple test, which can be quickly made on average samples selected from each lot of steel, provides assurance that the sheet is of the proper quality to make a high grade product.

The All-Welded Barge Steelweld*

E. H. EWERTZ†

PROGRESSIVE engineering demands economic gains in the promotion of new developments, and while time is required to fully protect such developments, they must bear evidence of leading to the ultimate goal and not to be just another opportunity for promotion and sale of material and equipment.

If radical departure from existing form or design is essential, the subject should be fully considered and necessary new designs developed along with a procedure that will accomplish the proper economic gain.

Such fundamental ideas governed the design and procedure of the all-welded steel deck barge "Steelweld."

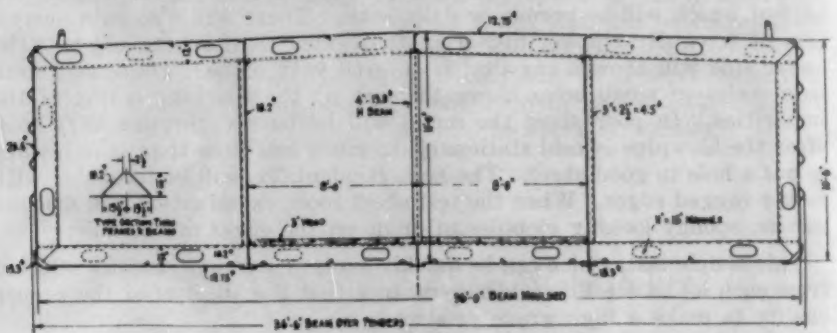


FIG. 1

This barge is 118 ft. long, 36 ft. beam, 10 ft. 6 in. high at side, and 11 ft. at center. When light the draft is 20 in. and when a 1000-ton cargo is carried the draft is 9 ft. 10 in.

The design, which embodies the Ewertz patents on special frame construction and flanged plating, takes into consideration the necessary allowances for welding stresses, the best condition for welding with a minimum amount of overhead work, and a simple way of assembling the many parts necessary to produce such complicated structure as a barge or ship.

Due to the varying stresses when loading, the wave stresses and harbor maneuvering, it was necessary to use heavy material and to fit not only three watertight transverse bulkheads but two watertight longitudinal bulkheads.

The side plating is ½ in. thick with 3½ in. flanges, deck plating is

*Paper to be presented at Annual Meeting, A.W.S., April, 1930.
†Consulting Engineer.

⅜ in. in narrow widths and having 3 in. flanges, and the bottom plating is 5/16 in. except at the ends, where it is increased to ⅜ in., this plating also having 3 in. flanges.

All bulkheads are ¼ in., flanged as shell plating and further reinforced with heavy angle stiffeners with toes welded to plating, thus forming with bulkhead itself, a stiffener like a channel.

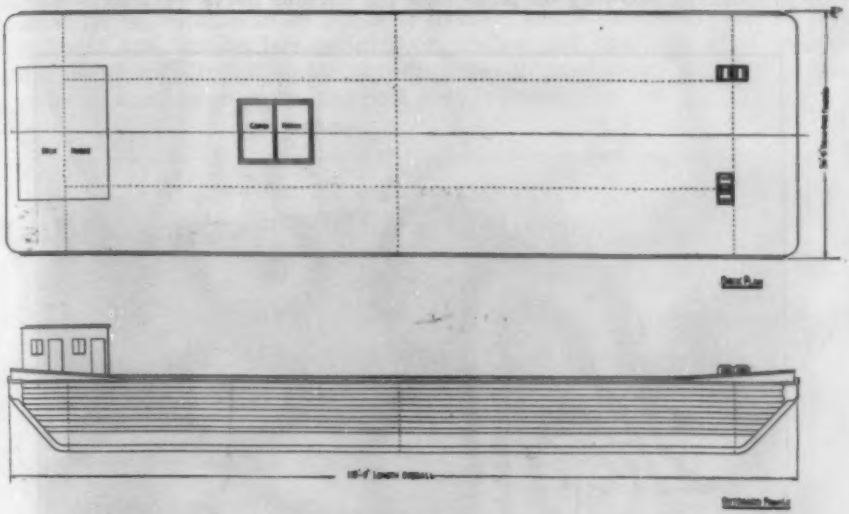


FIG. 2—OUTBOARD PROFILE

The frames on bottom, side and under deck are of flanged ¼ in. plates, formed like an inverted angle, the heel being 12 in. from shell plating and toes spreading 27 in. apart, and are provided with necessary openings for drainage, cleaning and painting. Construction at ends is extremely heavy with angle framing closely spaced, and stanchions and bracing of heavy angles. The end plate is ½ in. thick, reinforced with flat bars on edge, and the top and bottom joints of this plate are covered with 4 in. angles ½ in. thick.

The five fenders on each side are of ⅜ in. thick plates, pressed to proper form, and the inside of fenders are filled with wood and tar.

Deck rail is of ⅜ in. plating reinforced with flanged brackets 4 ft. apart and topped with a 2½ in. extra heavy pipe. The cargo hatch is reinforced to permit handling of heavy cargo, such as raw pig copper, and heavy stanchions support deck on sides of hatch. Similar stanchions are fitted at center throughout the whole length to further strengthen the deck.

The large steel deck house and cabin uses the patented special channel design (metal lumber). In the cabin a wood flooring is fitted, the sides and roof are insulated with 2 in. thick compressed cork. All windows, swinging and sliding doors are of steel with all brass hardware. The cabin is fitted with a coal stove and two standard pipe berths.

All deck fittings are of cast or forged steel, fitted on proper reinforcements. Pumps with proper piping and valves leading to all compartments permits water to be easily removed. Hatches give access to all compartments and the forced ventilation system prevents sweating.

The builder of this barge, the Electric Boat Company, of Groton, Conn., carefully planned the procedure to be followed in the various steps of production, showing on plans how the various parts were to be assem-

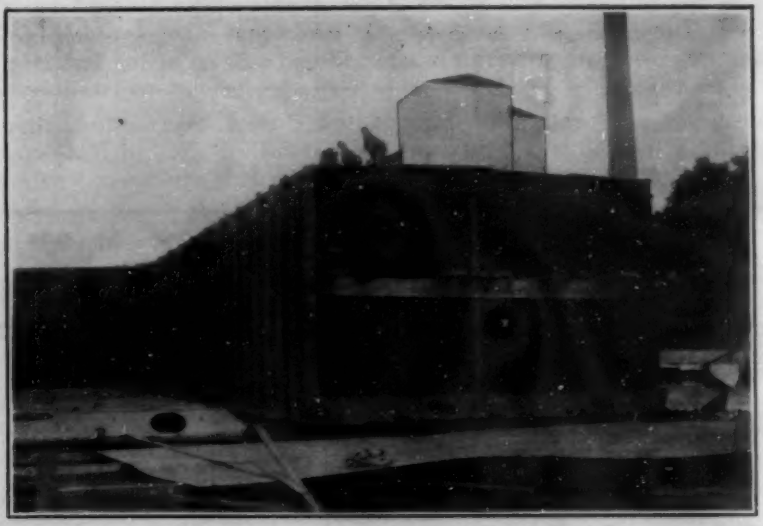


FIG. 3—ERECTION PROCEDURE OF FINISHED SECTIONS

bled and erected, and the detail plans specified the type and size of all welding, based on welding being equal to 8000 lb. per sq. in. stress. All welders were given a special course of training and each welder was compelled to pass the test prescribed by the American Welding Society. All of the 18,000 linear feet of welding on this barge was done by hand, using single operator units, there being no suitable automatic welding equipment obtainable at that time.

Nearly all the steel material, about 156 tons, was delivered to the assembly space, all flanged, by the Lukens Steel Company, and the necessary cutting was done by gas torch in the field. The only material that went through the fabricating shop was the end corner plates and the end angle frames, this material requiring forming.

The material was assembled in 18 major sections, permitting all welding to be done in the flat position, and then these 18 sections were lifted into proper position and welded together; and where parts could not move as a body, welds were made and peened as the work progressed. Practically all the steel was painted before assembly, except in the way of welded joints, thereby materially reducing rusting as well as cost of painting.

The erection of large, finished sections permits work to be so arranged that the valuable launching spaces are occupied for a very short period, compared to what is normal for riveted construction, thereby increasing the possible output of any plant.

Now, what is the economic gain obtainable by this new construction?

This barge, compared with other steel barges of the same carrying capacity, is from 25 to 30 per cent lighter, which means the use of less material and towing less weight, or towing and handling of a smaller size barge with its economic saving in handling expense, and due to less material and fewer parts, the first cost is lower.

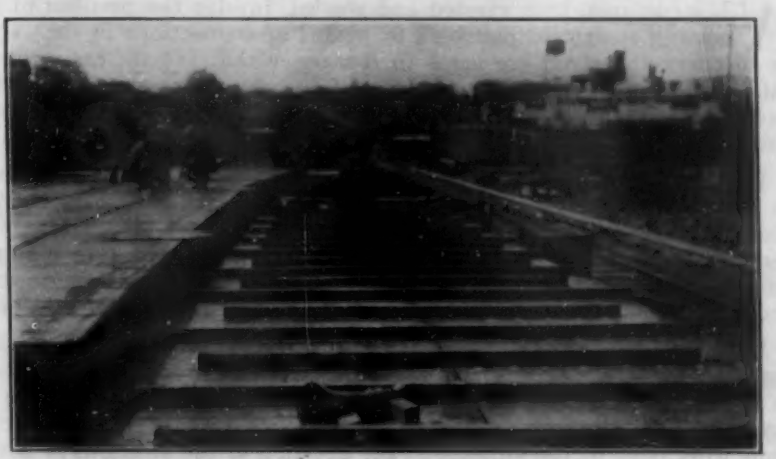


FIG. 4—DETAIL VIEW OF BULKHEAD WITH STIFFENERS TACK WELDED TO ONE PLATE ONLY

Again, the type of joints used eliminates many troubles from the standpoint of maintenance in that leaking and corrosion of rivets and similar defects costly to repair are avoided, and with the better distribution of material and a joint equal to at least 85 per cent of material joined, ordinary service will not disturb the joints and thereby eliminating the greatest expense in maintenance of steel barges.

Further, should the barge be seriously damaged, the repairing does not mean the removal of both damaged and adjoining parts, but by means of a gas torch the damaged parts may be cut out and new parts welded in, returning the barge to its original condition with much less loss of operation time.

By the division of the interior of the barge by watertight bulkheads it will float even though several compartments may be flooded due to collision or other serious damage.

In conclusion, we believe being justified in claiming to have contributed a new engineering feature, possessing economic advantages that will primarily be of benefit to the general public.

Strength Investigations on Welded Beam Connections*

CHARLES H. JENNINGS† and A. A. JAKKULA‡

INTRODUCTION

ALL structures, both riveted and welded, involve the problem of connecting structural members by means of connections in such a way as to carry the structure loads in a manner that is safe and economical. These connections, however, have often been the most unsatisfactory part of the design because only approximate methods and formulæ were used in their design.

In some structures, it has been found necessary to provide rigid beam to column connections in order to prevent distortion due to moment, to decrease deflections, and to produce economy. In other structures, it has been found desirable to use flexible connections in order to eliminate secondary stresses, and to eliminate highly statically indeterminate structures that cannot be readily analyzed by the simple laws of statics.

In many cases, the mere selection of a certain type of joint is not sufficient for a satisfactory connection since the location of the weld metal in relation to the structural members has a marked influence upon the flexibility and strength of any connection. The present paper refers to some work done by the authors on the question of the strength of flexible connections, and forms an introduction to this subject.

Design of Connections[1]

As mentioned in the introduction, this investigation was conducted for the purpose of studying the properties of various beam connections. The connection designs were based on a uniformly loaded floor beam such as is commonly used in practice. For convenience, an 8 in. I 18.4 lb. beam 8 ft. long was chosen for the tests and the design of the four types of connections investigated were based upon a uniformly loaded beam of this size.

The type I connections (Fig. 1) consisted of two ¼ in. fillet welds located midway between the beam flanges. The fillet weld lengths were calculated** to withstand the shearing force produced by the end reaction of a uniformly distributed load of such a value as to produce a maximum fiber stress of 18,000 lb. per sq. in. in the beam. No account was taken of the secondary bending moment produced by the restraint at the ends of

*Paper to be presented at Annual Meeting, A. W. S., April, 1930.

†Research Dept., Westinghouse Electric & Mfg. Co.

‡Temporarily connected with Westinghouse Research Department.

[1] These designs were suggested by Mr. G. D. Fish, Consulting Engr. of W. E. & M. Co., New York Office.

**Allowing 2000 lb. per linear inch for a ¼ in. fillet.

the beam. The other three types of connections contained the same size welds but varied in design in that two side angles were used and the welding detail was slightly different. Figure 1 shows details of the four types of connections.

Design and Preparation of Test Specimens

Two types of specimens were used in making the connection tests. One type consisted of an 8 in. I 18.4 lb. beam 8 ft. long mounted in a rigid frame. The connections used to join the test beams to their supporting columns were those under investigation. Figure 2 shows a detailed sketch of a beam specimen with a type I connection.

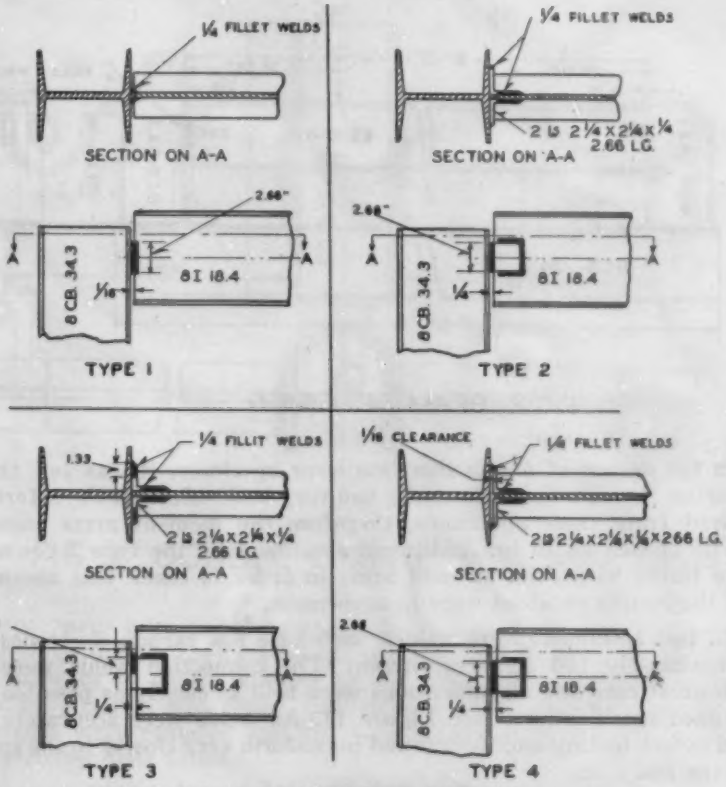


FIG. 1: BEAM CONNECTION DESIGNS

Although the connections were designed on the assumption of a uniformly loaded beam, such a loading was impractical from the testing standpoint, and consequently a two point loading was used. The points of load application for the two point loading were determined analytically (assuming free ends) so that the theoretical end slope for any given load on the beam was equal to the theoretical end slope of a similar beam with the same load uniformly distributed. The points of load application thus

obtained were found to be 0.211L (where L is the span) from the beam ends. The maximum bending stress produced by two concentrated loads located as figured above became 84½ per cent of the maximum bending stress produced by the same load uniformly distributed.

The second type of specimen used was of the double cantilever type as shown in Figure 3. These specimens consisted of two cantilever beams mounted opposite each other on a center column. The connections between the cantilever beams and the center column were the connections under investigation. This arrangement made a symmetrical specimen and thus produced in effect a connection between a beam and a rigid column.

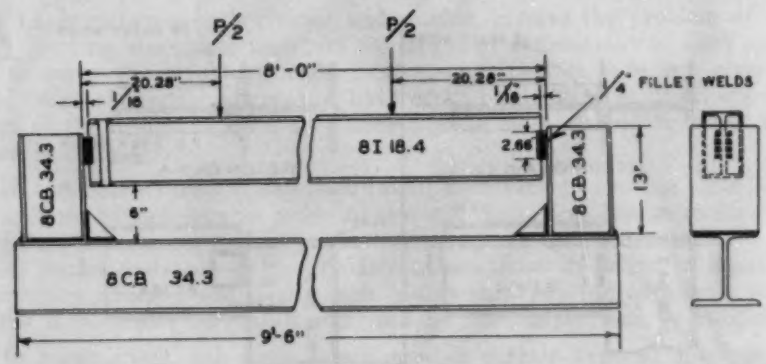


FIG. 2: BEAM SPECIMEN

In the design of the double cantilever specimen, it was felt that the shearing force on the connections had very little effect on the information desired from these specimens, therefore the moment arms were arbitrarily chosen as 16 in. Additional specimens of the type I connections were tested with 8 in. moment arms in order to check this assumption, and the results obtained were in agreement.

All test specimens were welded with bare low carbon electrodes using approximately 150 amperes current. The connection welds were made with great care and all dimensions were held as closely as possible to the designed specifications (see Figure 1). All welds were accurately measured before testing and were found to conform very closely to the specified lengths and sizes.

Description of Tests

A. BEAM SPECIMENS.—Six 8 ft. beam specimens (two specimens each for the type I, II and III connections), were tested. These specimens were tested in transverse bending by means of a 400,000 lb. Olsen testing machine. The loads were applied at the designed loading points by means of an intermediate loading beam and loading blocks.

Four dial-gages were mounted at one end of the beam on specially located bars welded from the top to the bottom flanges on both sides of

the beam. (See Figure 4.) By means of these dial-gages it was possible to measure the actual end slope of the beam with respect to the column.

Four dial-gages were also mounted below the beam, one at each end and one under each loading point, in order to measure the beam deflection at the loading points (see Figure 4).

The beam surfaces and connections were painted with a coating of cement before testing. This coating was very brittle and cracked off at points of yielding and gave a visible means of detecting yielding. Very good results were obtained from this cement coating and a number of photos are shown later in the paper.

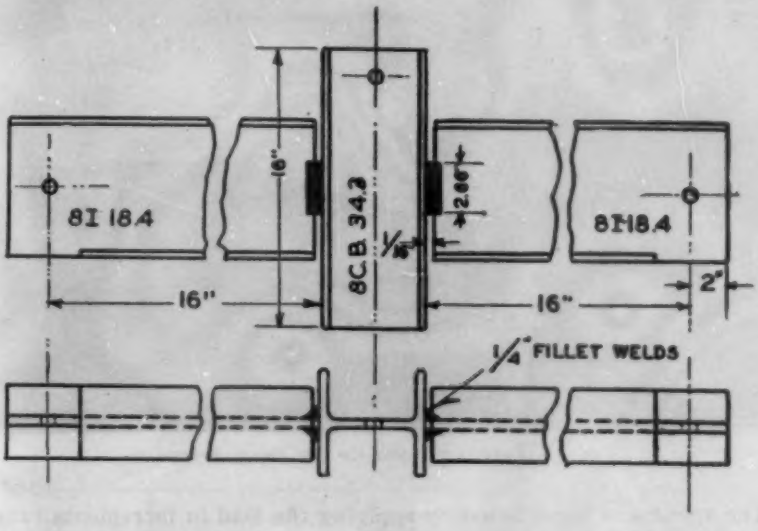


FIG. 3 · DOUBLE-CANTILEVER SPECIMEN

The specimens were tested by applying the load in 2000 lb. increments. Several times during the process of loading the load was completely removed in order to detect whether or not yielding had occurred in the connection. After it was found that definite yielding had occurred, the instruments were removed and the load continuously applied until the beam or connections failed.

B. DOUBLE CANTILEVER SPECIMENS.—Four types of connections were tested with specimens having 16 in. moment arms, and one type of connection was tested with specimens having 8 in. moment arms. Two specimens of each type of connection were tested, and because each specimen contained two connections, the information obtained was equivalent to making four tests of each connection.

These specimens were tested in transverse bending in a 20 ton Amsler hydraulic testing machine. The loads were applied by means of a loading beam and a system of connections as shown in Figure 5.

Dial-gages were mounted next to the columns on the top and bottom flanges of each arm by means of specially designed clamps. These clamps were constructed in such a manner as to locate the gages in the vertical axis of the beam web.

The small lugs under the gages (see Figures 5 and 16) were placed on the specimens to facilitate measuring the angle by means of Huggenberger Tensometers. It was found, however, that the range of these instruments was too small, so they were not used.

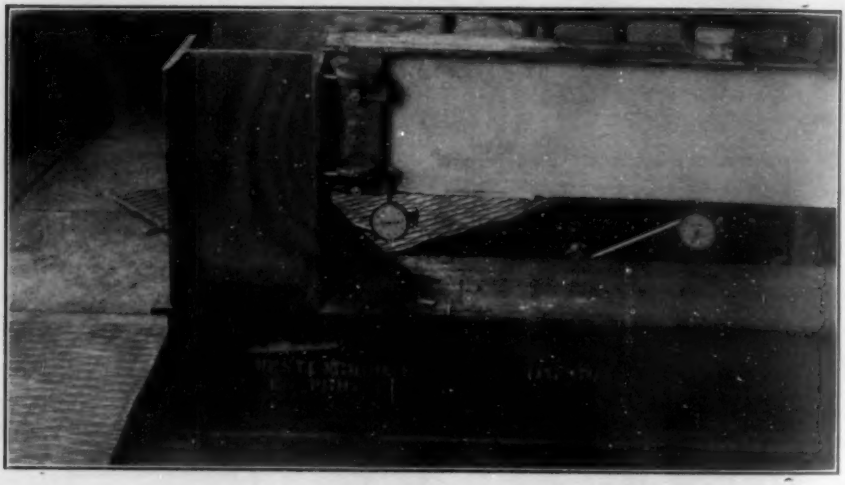


FIG. 4—INSTRUMENT SET-UP FOR BEAM SPECIMEN

The specimens were tested by applying the load in increments ranging from 10 to 50 lb., depending upon the type of connection. Several times during the process of loading, the load was completely removed in order to detect as closely as possible the loads to cause yielding. After yielding was obtained, the instruments were removed and the load was continuously applied. It was impossible to test the connections to destruction by this set-up.

Test Results.

The results obtained from the various tests were tabulated in the form of curves in order that they could be easily interpreted. In Figure 6, the measured end slopes of the beam specimens with type I, II and III connections were plotted against the total load on the specimen. In Figure 7, the deflections at loading points for beams with type I, II and III connections were plotted against the total load on the specimen.

Figure 8 shows the test results obtained from two beam tests with the type II connection. These curves were typical of those obtained for all three types of connections. It will be noted that very close agreement was obtained between the two tests.

In Figure 9, the end slopes of the various types of connections as

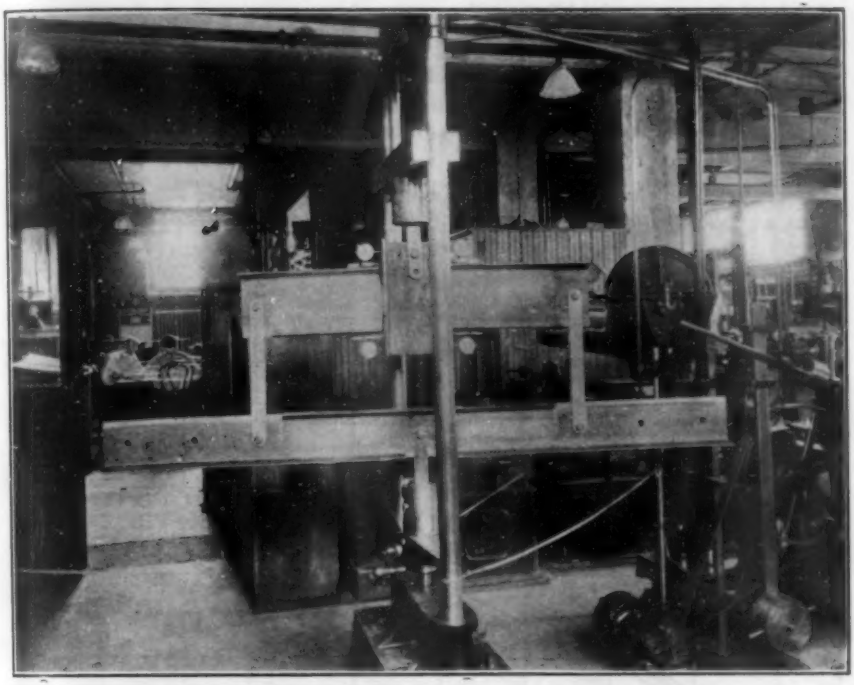
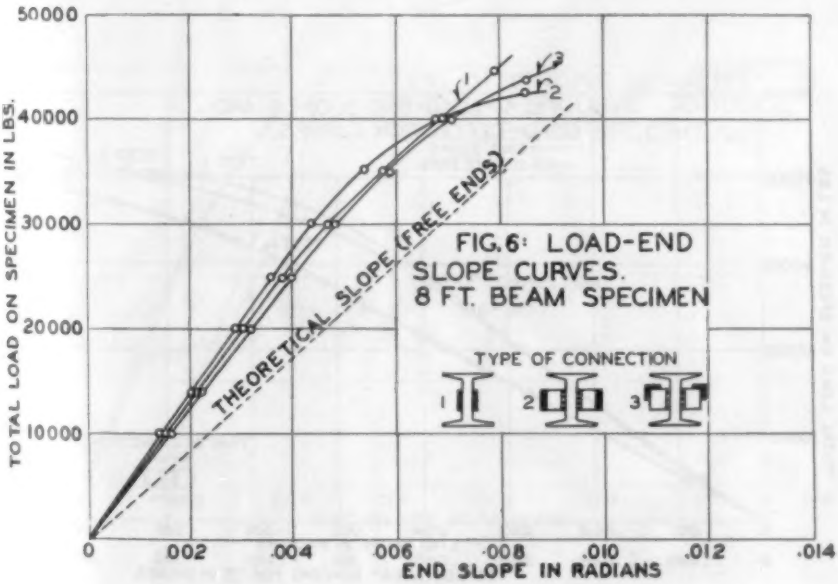


FIG. 5—DOUBLE CANTILEVER SPECIMEN MOUNTED FOR TEST



obtained from the double cantilever specimens were plotted against the bending moments on the connection.

A complete moment-angle curve of the type I connection (16 in. moment arm) is shown in Figure 10. The moment-angle curve for the type I connection as obtained from the 8 in. moment arm specimens is similar to Figure 10.

Comparative stress values for beam specimens with different connections at loads to cause yielding and final failure are given in Figure 11.

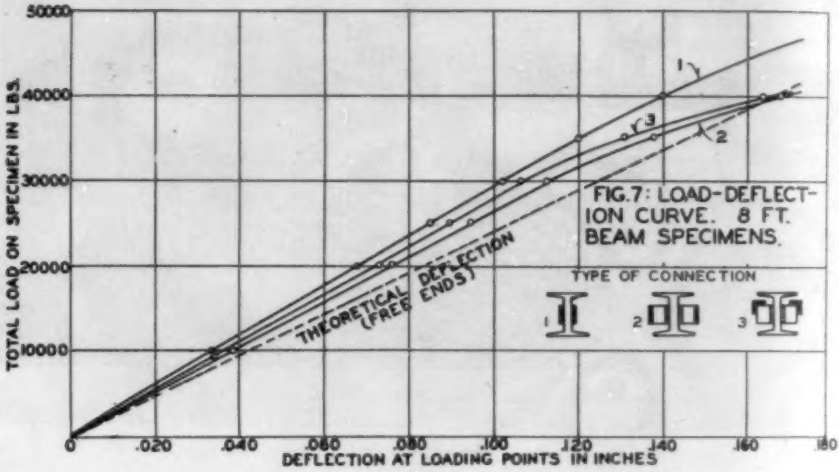
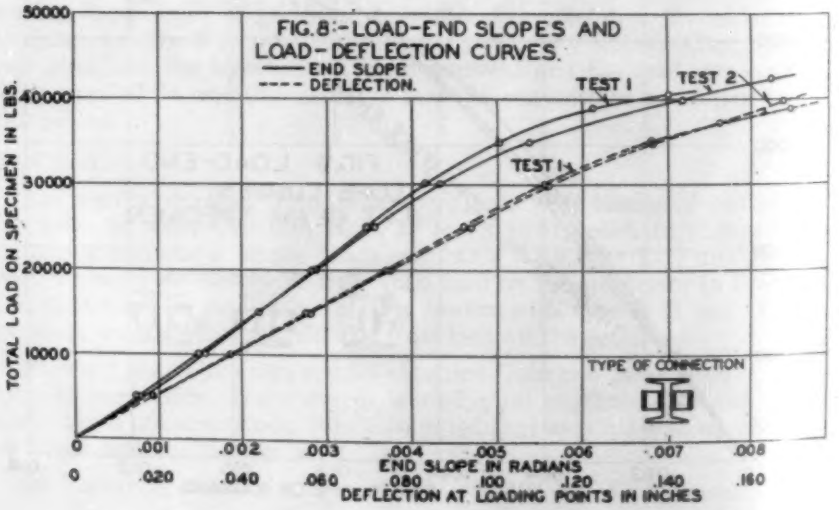


FIG. 7: LOAD-DEFLECTION CURVE. 8 FT. BEAM SPECIMENS.



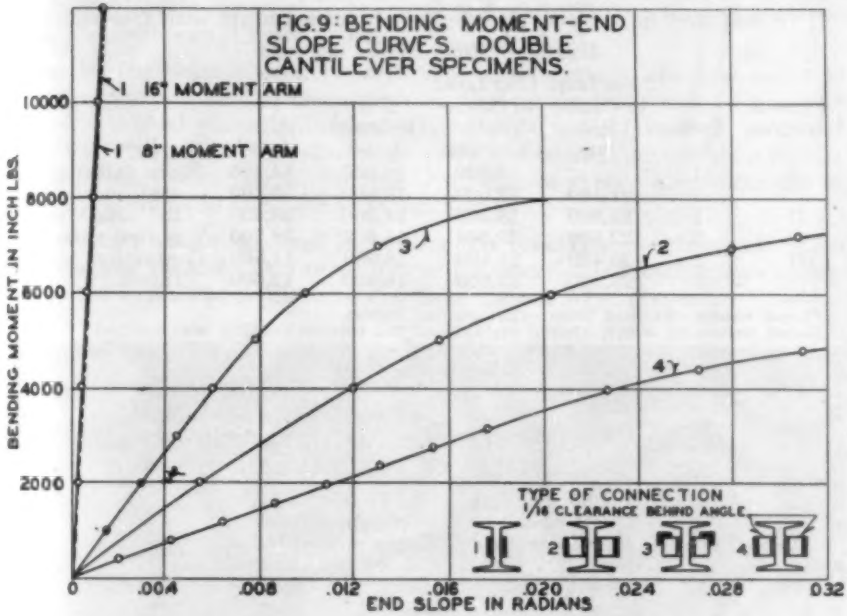


FIG. 9: BENDING MOMENT-END SLOPE CURVES. DOUBLE CANTILEVER SPECIMENS.
16" MOMENT ARM
8" MOMENT ARM
BENDING MOMENT IN INCH LBS.
10000
8000
6000
4000
2000
0
.004
.008
.012
.016
.020
.024
.028
.032
END SLOPE IN RADIANS
TYPE OF CONNECTION
1/16 CLEARANCE BEHIND ANGLE
1
2
3
4

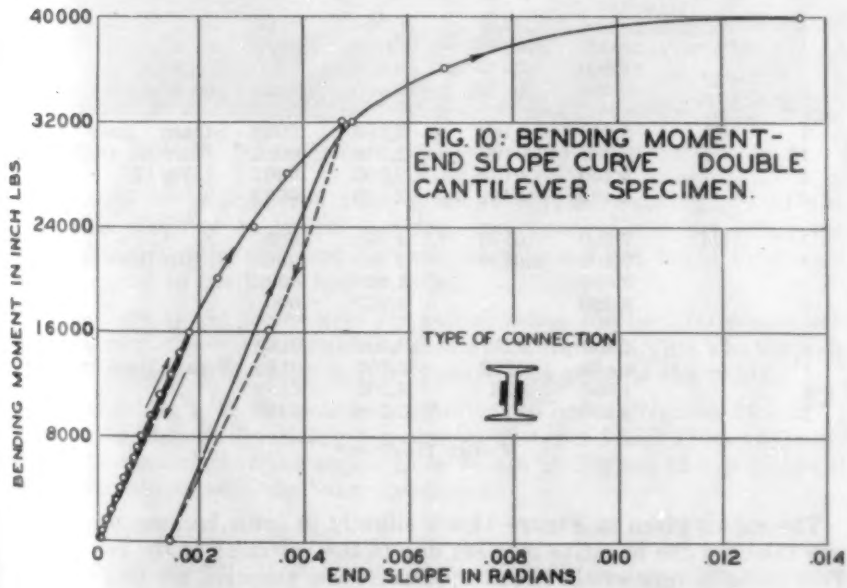


FIG. 10: BENDING MOMENT-END SLOPE CURVE DOUBLE CANTILEVER SPECIMEN.
BENDING MOMENT IN INCH LBS.
40000
32000
24000
16000
8000
0
.002
.004
.006
.008
.010
.012
.014
END SLOPE IN RADIANS
TYPE OF CONNECTION

STRENGTH DATA ON 8-FT. BEAM SPECIMENS

(Computed on the basis of a uniformly loaded beam with free ends)

Maximum Fiber Stress in Beam

Type of Connection	Spec. No.	*For Load to Cause Yielding	†For Load to Cause Visible Yielding	For Designed Load	For Ult. Load	Remarks
I	1	.....	16,800	18,000	54,400	Beam failed by lateral buckling due to long unsupported span.
	2	21,100	22,000	18,000	50,500	
II	1	23,300	19,400	18,000	38,400	
	2	22,800	20,300	18,000	38,700	
III	1	25,400	21,100	18,000	44,500	Connection welds failed.
	2	25,200	29,600	18,000	42,000	

*Load values obtained from experimental curves.
†Load values at which visible yielding of the cement coating was noticed.

FIGURE 11—TABLE

STRENGTH DATA ON CONNECTIONS AS OBTAINED FROM DOUBLE CANTILEVER SPECIMENS

Connection No.	Type of Con.	Visible Yielding (Cracking of Cement) Bend Moment	Visible Yielding (Cracking of Cement) End Slope	Yielding as Obtained from Experimental Data Bend Moment	Yielding as Obtained from Experimental Data End Slope	Remarks
1	I	28,000	.00410	16,000	.0019	Strain lines started in beam web on com. side.
1′		28,000	.00410	16,000	.0019	
2	16″ arm	28,000	.00410	16,000	.0019	
2′		32,000	.0053	16,000	.0019	
1	I	28,000	.0044	16,000	.0020	See Fig. 16.
1′	8″ arm	28,000	.0044	16,000	.0020	See Fig. 16.
2		24,000	.0039	16,000	.0020	See Fig. 16.
2′		30,000	.0049	16,000	.0020	See Fig. 16.
1	II	5,200	.0163	2,200	.0062	Strain lines across face of angle. See Fig. 17.
1″		5,200	.0163	2,200	.0062	
2		4,400	.0132	2,200	.0062	
2′		4,400	.0132	2,200	.0062	
1	III	7,200	.0140	4,000	.006	
1′		7,200	.0140	4,000	.006	Strain lines in angle.
2		8,000	....	4,000	.006	
2′		8,000	....	4,000	.006	
1	IV	4,400	.0272	3,300	.0182	
1′		4,400	.0272	3,300	.0182	Strain lines in angle.
2		4,000	.023	3,300	.0182	
2′		4,000	.023	3,300	.0182	

FIGURE 12—TABLE

The values given in Figure 11 are slightly in error because no account was taken of the negative moment due to the restraint of the connection. This error is very small, however, because the negative bending moment is small in comparison to the maximum bending moment due to the beam load.

Figure 12 gives comparative values of bending moments and end slopes at which yielding occurred in the various types of connection as obtained from the double cantilever results.

For all the beam specimens with type I connections, the first signs of failure appeared in the beam web on the compression side. Figure 13 shows a typical shear failure of a beam web as obtained on a type I connection test. Figure 14 shows the same specimen after complete failure of the beam due to lateral bending. No signs of cracks are noticeable in any of the welds in the type I connections.

The beam specimens used in testing the type II connections all failed in a manner similar to those with type I connections. Yielding was apparent in the welds but no signs of cracks or failure were present.

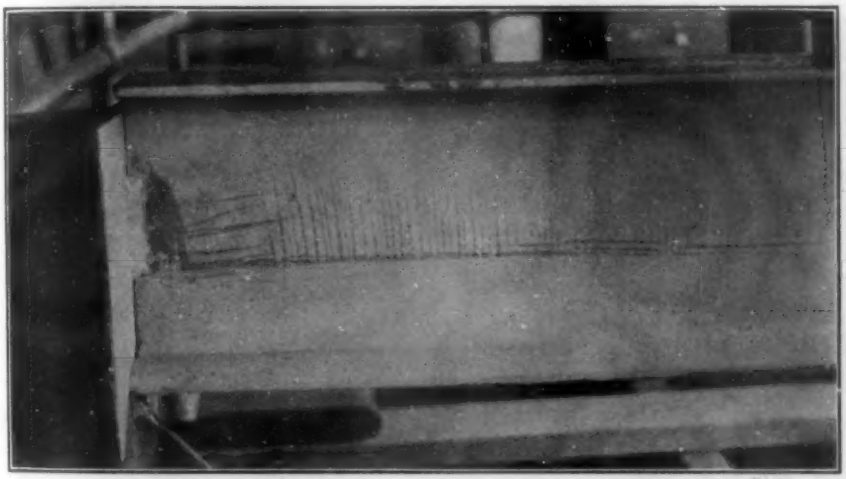


FIG. 13—SHEAR FAILURE OF WELDED BEAM CONNECTION

It should be remembered that although the ultimate failure of the specimens was due to lateral bending, this bending did not affect the experimental results obtained because bending did not begin until long after yielding in the beam had occurred.

The failure of the beams used for investigating the type III connection occurred in the connection welds as shown in Figure 15. This was the *only* type of connection in which definite failure took place in the welds.

In connection with the double cantilever specimens, Figures 16 and 17 show typical types of yielding for connection types I and II respectively. It will be noticed that the strain lines shown in Figure 16 are identical to those obtained with the beam specimens.

Photo Elastic Investigations

In connection with the strain lines obtained on the cement coated surfaces around the type I connection, it was noted that these lines always started on the compression side of the web and progressed parabolically

FIG. 14—BEAM SPECIMEN AFTER FAILURE

FIG. 15—FAILURE OF TYPE III CONNECTION

toward the upper end of the connection (see Figure 16). In an effort to obtain further information upon these lines, it was decided to make a photoelastic test on a similar connection by means of a bakelite model. This test was conducted by R. V. Baud* and some very interesting results were obtained.

The specimen used in making this test is shown in Figure 18, and is modeled from a double cantilever specimen. In the design of this specimen, it was assumed that although the I-beam flanges had some stiffening effect that this effect was very small in the neighborhood of the weld and could be neglected.

The 8 in. I-beam was therefore represented by a rectangular piece of bakelite. The height was taken equal to 2 in. and all other important dimensions reduced accordingly. An extension CDEF of the bakelite model was provided in order to clamp the model between two steel plates S. These steel plates projected over the bakelite piece to points AB in order to represent the condition of the welds. Although the initial stresses produced by this arrangement were different than those produced by actual welds, the lateral stresses due to loading were about the same. Therefore the superimposed lateral stresses in the actual weld did not differ much from those in the set-up. A photograph taken of this specimen is shown in Figure 19. The curves of maximum lightness and darkness on this photo represent lines for which the maximum shearing force is constant.

A close examination of Figure 19 shows results strikingly similar to those obtained from cement on actual steel specimens. It will be noted that on the compression end of the joint four dark lines are present while on the tension end there are only three dark lines. This corresponds to results obtained on steel specimens in that it indicates the presence of higher stresses on the compression end. Also the lines obtained in the bakelite model have the same characteristic shape as those obtained in the steel specimens.

An analytical study of the shearing stresses as obtained in the photoelastic test indicated that they were very high and could not be calculated by ordinary beam theory. This study, however, will not be included in this paper.

Discussion of Experimental Results.

In discussing the experimental results obtained from the beam and double-cantilever specimens, it is desirable to first consider those obtained from the double-cantilever specimens.

From the double-cantilever tests information regarding the bending moment and end slope for the various types of connections was obtained as shown in Figure 9. The bending moments to produce yielding in the various connections were also obtained. (See Figure 12). The difference between the bending moments to cause yielding as determined by the cement cracking and the experimental data, is due to the fact that there is a delay between the yielding in the metal and the cracking of the coating. The thickness of the cement coating has a great effect upon this delay.

*Research Engineer, W. E. & M. Co.

The curves given in Figure 9 show the relative flexibilities of the different connections. Curves of this type, however, are of more importance than to merely give a comparison of connection flexibility, for they can be used as a basis for designing beams of any span and loading. An example showing the possibility of using moment-end slope curves in this manner is included under Appendices I and II.

From the beam specimen tests information regarding the beam deflections, the beam end slopes, and the types of ultimate failure were obtained.

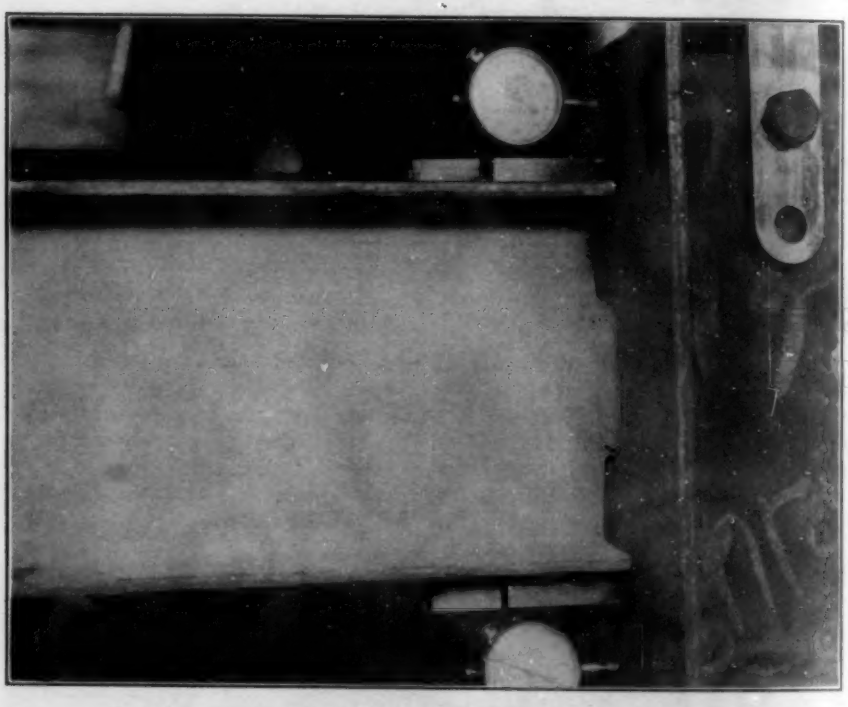


FIG. 16—INSTRUMENT SET-UP FOR DOUBLE CANTILEVER SPECIMEN

It will be noted that the load deflection curves shown in Figure 7 are grouped together and approach very closely to the theoretical deflection curve as figured on the basis of free ends. (The maximum variation from the theoretical curve was obtained for the type I connection and this variation was about 19 per cent). The fact that the curves were grouped so closely to the theoretical curve was confirmatory evidence that these types of connection were very flexible. The negative end bending moment produced by the angular rotation of all the connection investigated were small in comparison with the maximum bending moment of the beam and therefore had little effect on the deflection of the beams.

In regard to the load-end slope curves shown in Figure 6, it will be noted that the experimental curves are again grouped closely together

but that there is a large difference between them and the theoretical end slope as figured on the basis of free ends. (In the case of the type I connection, it will be noted that the experimental curve varies more than 30 per cent from the theoretical curve). The larger per cent of variation from the theoretical curves is explained by the lack of rigidity of the supporting columns.

The end-slopes as measured only took into account the angular rotation of the beam ends with respect to the supporting column and did not account for any angular rotation due to the flexibility of the columns themselves. Consequently, the greater the flexibility of the column, the

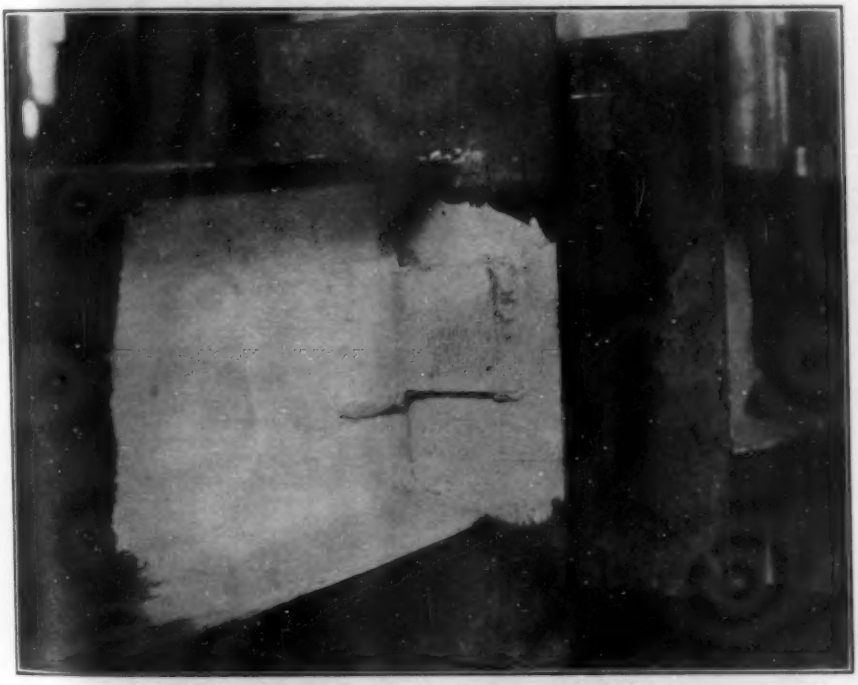


FIG. 17—STRESS LINES ON TYPE II CONNECTION (DOUBLE CANTILEVER SPECIMEN)

smaller was the measured end-slope for a given load. Because the flexibility of the supporting columns has such a great effect upon the end-slope of the beam, the greatest of care must be exercised in the interpretation of results obtained from beam specimens.

An example of how results of beam specimens with semi-rigid columns can be misinterpreted is illustrated in Figure 11. The results in Figure 11 show that no yielding occurred in the connections until after designed load of the beam had been reached. If the beam columns had been rigid in the tests, the results would have been entirely different and it is quite possible that yielding would have occurred before the designed load was reached. Consequently, even though the connection proved satisfactory

under the conditions of the writer's tests, the results cannot be considered typical for all cases of column rigidity.

The fact that the two methods used to detect yielding agreed so closely in the beam tests was due to the use of a less sensitive instrument set-up.

The types of failures obtained for the various connections were also of extreme importance for they indicate the actual types of failures that would occur in the various connections.

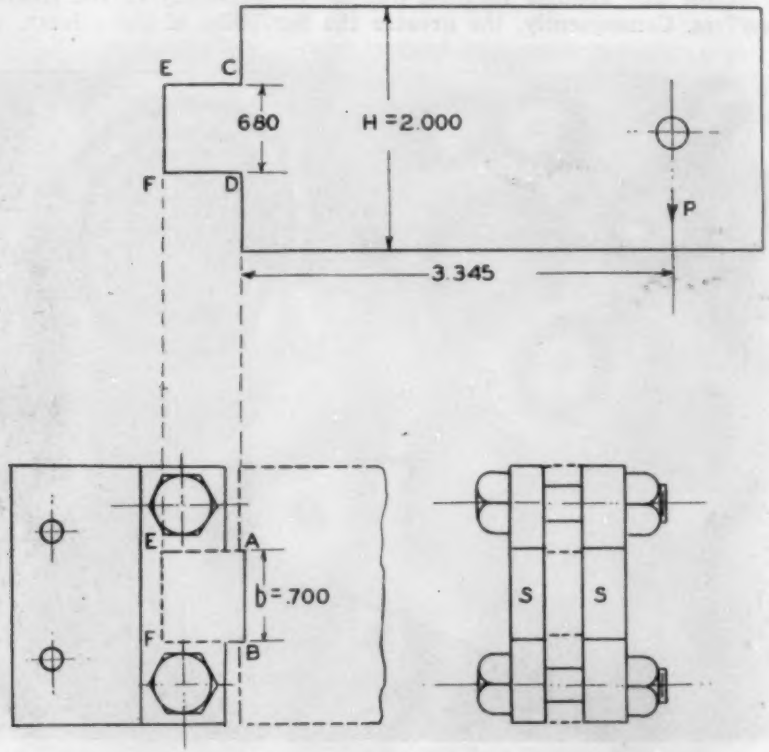


FIG. 18: PHOTOELASTIC SPECIMEN.

It was found in the tests of the type I connections that no failures took place in the welds. The fact that no failures took place in the welds in the writer's tests can possibly be explained by the fact that the critical section of the weld metal was larger than that of the joining portion of the beam web. This necessarily caused the first yielding to take place in the beam web which was ductile and could yield and relieve excessive stresses.

For the specimens tested, the welds and the beam spans were short. Variations of the weld sizes, the beam sizes, and the beam spans might produce entirely different conditions, therefore no general conclusions as

to the properties of this type of connection can be drawn from the few tests innumerated herein.

The type II connections proved to be very satisfactory both from the standpoint of flexibility and strength for here again no failures took place in the welds.

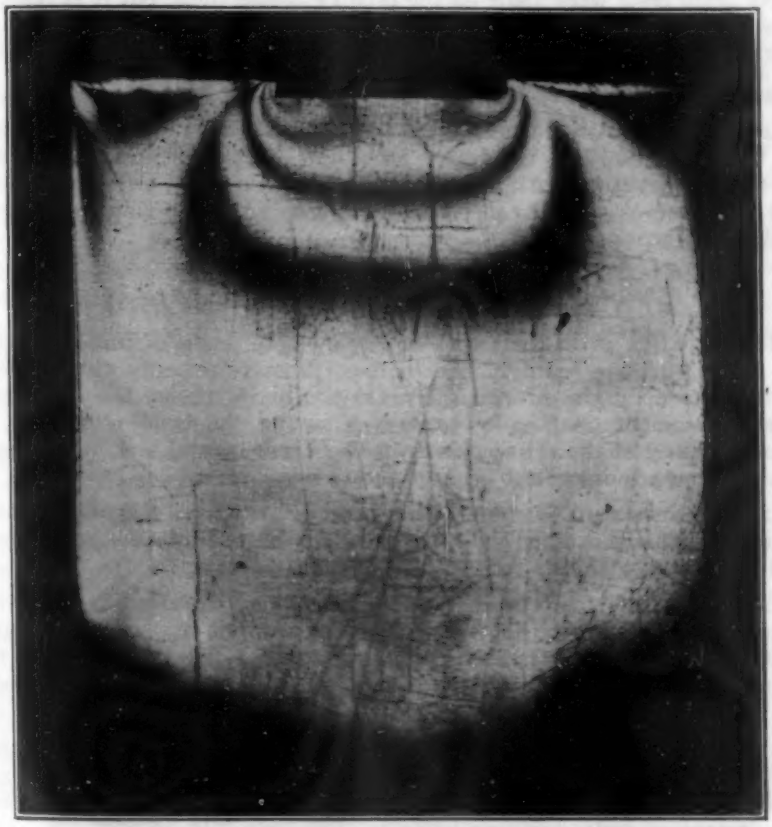


FIG. 19—PHOTOELASTIC PICTURE OF STRESSES IN TYPE I CONNECTIONS

The fact that no failure took place in the welds was due to the great flexibility of the side angles. No doubt there is a bending moment present in the welds due to the twisting of the angle caused by the end-slope of the beam, but this moment is not a serious factor and is relatively small if the angle leg is long in comparison to its thickness.

In the type III connection, the welds on the tops of the side angles failed. This was explained by the fact that the weld on the top of the angle greatly reduced the flexibility of the angle. The reduction of the angle flexibility caused a high bending moment to occur in the weld and thus caused them to fail.

As mentioned in the discussion of the type I connections, however, the meager data obtained by the authors is not enough to draw general conclusions for all designs of the type II and III connections. It is safe, however, to assume that wherever angles are used to produce flexibility, they should always be welded on the end of the angle legs and never on the sides.

No beam tests were made on the type IV connections, but it is felt that this type would produce results more satisfactory than those obtained from the other three types of connections.

Practical Application of Test Results

Information on weld connections as obtained from test specimens similar to those described herein are of great use to the designer of weld fabricated structures. An example of how the double-cantilever moment-end slope curves (Fig. 9) may be used is given in the following equation as derived in Appendix I.

$$M = \frac{Kql^2}{12\,(2EI + K)}$$

This equation gives the connection moment in terms of the beam span, the beam load, the beam moment of inertia, and the connection constant K. Therefore by the proper choice of the above items, the connection bending moment can be determined. Knowing the bending moment to cause yielding in the connection from the experimental test, it is readily evident whether or not the beam connection will be safe.

If the beam's span, load and size are necessarily fixed the beam connection can be changed (thus changing the value K) until the desired moment M is obtained.

In the case of connections in which the stress distribution is known, equations similar to the above can be developed, but in place of the equation being in terms of the end bending moment, it will be in terms of the connection stress. An equation of this kind for the type IV connections is*

$$S_t = \frac{Kql^2}{24\,\alpha\,bc^2\,(2EI + K)} \qquad (S_t = \text{maximum shear stress in connection angle})$$

Equations of the above type may be found to be of very great help in the designs of welded connections for they take into account the actual flexibility of the connections and eliminate the use of many arbitrary assumptions which may lead to large errors.

Conclusions

The writers have not attempted to give the impression that the types of connections described in this report are necessarily the only connections that could be used. They have merely attempted to investigate various connections employing only enough weld metal to carry the end shear, and to arrange the weld metal in such a manner as to obtain different degrees of flexibility. In an effort to obtain the above information, they have tried to develop satisfactory methods for obtaining the required

*See Appendix II.

information and to show how the obtained information can be of practical use to designers.

The types of connections that are used in practice depend upon the location and the methods of fabrication. It is obvious, therefore, that welding the web of beams directly to the face of columns or other beams would rarely be used because of the difficulty of cutting beams to exact lengths.

When joining beams to columns, it is common practice to use seat angles below and clip angles above. The arrangement gives a connection that is favorable from nearly every point of view. However, when a beam frames in between two other beams, it is sometimes impossible to use seat angles because of the lack of available space. This leads directly to the use of the type II, III and IV connections.

It is also desirable to have flexible connections for beams of this type in order to keep the cross beams from twisting the beams into which they frame. The information obtained on the type II, III and IV connections is therefore of real practical value.

The results obtained from the various tests can be briefly summarized as follows:

1. The method of testing connections by means of large beam specimens furnish actual results as to the type of failure of the connection. Careful consideration must be used in the interpretation of results from this type of specimen, however, because the supporting columns of the beam are not perfectly rigid.

2. The results from the beam tests on the type II and III connections indicated that whenever angles are used in connections to produce flexibility, welding should never be placed on the sides of the flexible leg of the angle.

3. The most flexible connection was obtained by the type IV connection where a small clearance was left between the side angles and the supporting column.

4. The safety of a connection made by welding a beam web directly to a column lies in the fact that the critical section of the weld is larger than the section of the beam web to which the weld joins. Connections of this type might be satisfactory for static loads but they should never be used for dynamic loads. Also, the fact that the connection herein investigated did not fail is no proof that all connections of this type would be satisfactory.

5. The method of testing connections by means of double-cantilever specimens furnish valuable information as to the joint flexibility.

6. The data obtained from double-cantilever type specimens can be used as a means of designing beams and connections, as shown in the discussions.

7. The double-cantilever specimens furnish a method for investigating the stress distribution in different types of connections.

As mentioned in the first of the paper, the present tests are merely an

introduction to the large volume of work that can be done along this line in an effort to improve the efficiency and strength of welded structures. It is hoped, therefore, that in the future, additional work will be done on other types of connections.

Acknowledgments

The writers wish to give particular acknowledgment to J. M. Lessells, in Charge of Mechanics Section of the Westinghouse Research Department, and Dr. S. Timoshenko and G. D. Fish, Consulting Engineers for Westinghouse Electric and Manufacturing Company, for valuable aid and suggestions in conducting this investigation.

Appendix I.

The development of an equation for designing a uniformly loaded beam using a connection for which a moment-end slope curve has been obtained by the use of a double-cantilever specimen, is as follows:

The angular end-slope for a uniformly load beam of span as computed on the assumption of free ends equals

$$\Phi = \frac{dy}{dx} = \frac{q^3}{24EI} \quad \text{where q is the load per unit length} \qquad (1)$$

The negative end slope that would be produced by a bending moment M acting at the beam ends due to the restraint of the connection equals

$$\Phi = \frac{M}{2EI} \qquad (2)$$

The actual end slope of a uniformly loaded beam with a total load q and an end bending moment M would be

$$\Phi = \Phi_1 - \Phi_2 = \frac{q^3}{24EI} - \frac{M}{2EI} \qquad (3)$$

From the experimental moment-end slope curve of the connection, it is possible to obtain M as a function of ϕ

$$M = f(\phi)$$

For the elastic stage

$$M = K\phi \qquad (4)$$

where K is a constant which may be obtained from the moment-end slope curve.

Therefore

$$\frac{M}{K} = \frac{ql^3}{24EI} - \frac{M}{2EI}$$

$$M = \frac{Kql^3}{12\,(2EI + K)} \qquad (5)$$

Where M = the bending moment on the connection in in. lb.
l = the beam span in inches
q = unit load on the beam in lb.
K = experimental constant
I = moment of inertia of the beam

Similar equations can be developed for beams with different load arrangements.

If the stress distribution in the connection is known, Equation 5 can be expressed in terms of the stress rather than in terms of the bending moment. The equation in either form, however, makes a very accurate and convenient method for designing welded beams and connections.

APPENDIX II.

The development of an equation for designing a uniformly loaded beam using a type IV connection (see Fig. 1) for which a moment end-slope curve has been obtained by the use of a double-cantilever specimen is as follows:

The derivation of the equation for the connection bending moment M is the same as that given in Appendix I.

$$M = \frac{Kql^2}{12\,(2EI + K)} \tag{6}$$

where K is the experimental constant obtained for the type IV connections.

In order to have the equation in terms of the stress in the connection, it is necessary to obtain the relation between the connection stress and the connection bending moment. An analysis of the type IV connection which has a clearance between the angle and the column shows that each angle leg can be considered as a rectangular bar in torsion. The maximum stress in a rectangular bar in torsion can be obtained by the approximate formulæ*,**

$$S_t = \frac{M'}{\alpha\, bc^2} \tag{7}$$

where M' = the bending moment on the bar
b = the depth of the bar
c = the thickness of the bar
α = a constant depending upon the ratio b/c*
S_t = maximum shear stress in the bar.

In the case where two angles are used as in the type IV connection, M' equals half the connection moment M.

The equation therefore becomes

$$M = 2S_t\, \alpha\, bc^2$$

Substituting this value of M into Equation (6) the stress in the angle equals

$$S_t = \frac{Kq^3}{24\, \alpha\, bc^3\,(2EI + K)} \tag{8}$$

This gives an equation which can be used to determine the actual stress in the beam connection.

*Applied Elasticity, by Timoshenko and Lessells, p. 33.

**Stresses in the connection angles computed by Equation (7) for bending moments causing visible yielding in the angles agreed very closely to the experimental results.

The Economics and Advantages of Pipe Welding with Stock Fittings*

By WILLARD P. CURLEY†

THE use of TUBE-TURNS, the seamless drawn fittings for pipe welding, has been standard practice for years in Europe and while by no means a new departure in this country, is yet young enough to be considered one of the latest developments broadening the field for the welding industry and for the benefit of those not entirely familiar with them, this presentation is intended to point out some of the features which make them ideally adapted to a better welded piping job.

Tube-Turns are fittings for the welded installation. They are designed for welding, so that much of our efforts are directed with a view

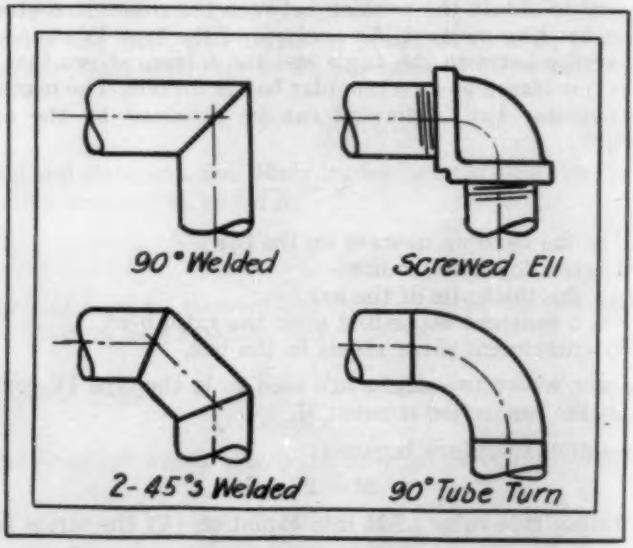


Fig. # 1 Four types of 90 deg. elbows were included in the experiments.

towards promoting the development of pipe welding and it has been gratifying to note that the rise in the popularity of this method of pipe fabrication has been coincident with the progressive perfection of

*Paper to be presented at Annual Meeting, A. W. S., April, 1930.
†Manager, New York District, Tube-Turns, Incorporated.

these fittings and other products for the welded job, such as the recently introduced high hub welding neck flanges.

Tube-turns are short radius fittings. They are not made by a bending process but by forging. Straight lengths of seamless tubing are forced

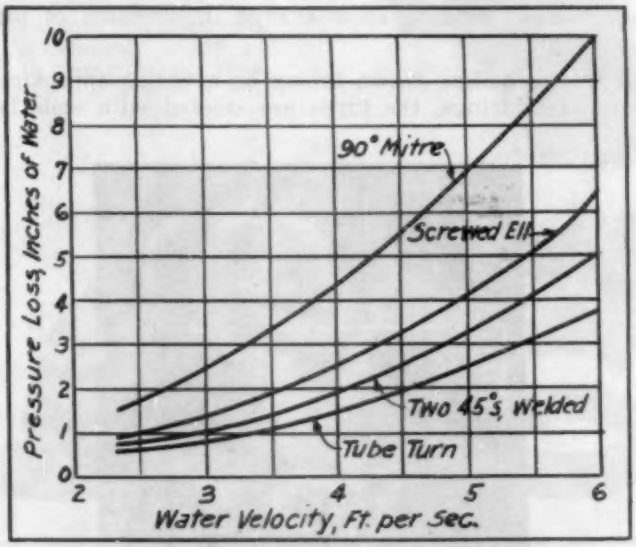


Fig. 2 Shows the relative advantages of the four turns.

FIG. 3—TUBE-TURNS IN INSIDE OVERHEAD PIPING. QUICKLY WELDED—A SMALL AMOUNT OF CLEARANCE NECESSARY—AND WHEN IN, THEY ARE IN TO STAY

over a horn shaped mandrel or die by hydraulic pressure at a forging temperature. The resulting product is unique in providing all of the following favorable features:

1. Uniform thickness of wall.
2. Absolutely circular cross-section.
3. Perfect smoothness of surface.
4. Minimum possible bending radius.
5. The same weight per unit as the pipe in the line.
6. Ease of assembling.
7. Production of bend to allow change in direction of pipe at any design angle.

Formed from seamless drawn tubing on a radius approximately that of standard cast fittings, the turns are stocked with ends beveled for

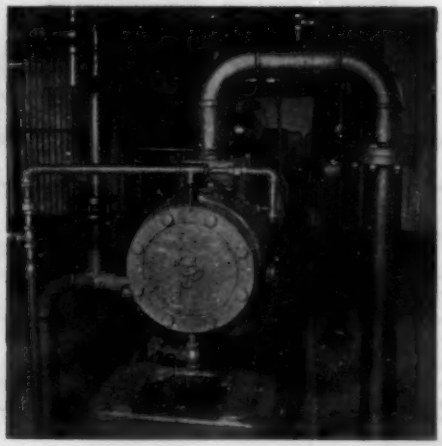


FIG. 4—TUBE-TURNS IN SUCTION AND DISCHARGE CONNECTIONS TO GAS COMPRESSOR

FIG. 5—90 DEG. TUBE-TURNS—CHANGING GRADE AND DIRECTION OF PUMP HOUSE PETROLEUM LINE

welding in 45 deg. elbows, 90 deg. elbows and 180 deg. returns in sizes 1 in. through 18 in. in the standard iron pipe sizes and 1½ in. through 12 in. in extra heavy iron pipe sizes. They correspond exactly in outside diameter, inside diameter and wall thickness with the iron pipe sizes.

FIG. 6—TUBE-TURNS ARE FAR LIGHTER IN WEIGHT—AN 8 IN. TUBE-TURN WEIGHS 68 LB.; WHEREAS, THE WEIGHT OF AN 8 IN. FLANGED FITTING, WITH ACCESSORIES, IS 312 LB. IN THIS CASE, THE TUBE-TURN IS ALMOST 4/5THS LIGHTER

For example, the O.D., I.D. and wall thickness of a 6 in. turn is the same as a 6 in. pipe. The material used in making the stock Tube-turn is a mild carbon steel and specials may be made of brass, copper, toncan iron, aluminum and various stainless steel alloys, etc.

Stock Tube-turns are formed to a radius of one and one-half times the nominal pipe diameter—thus: The radius and center to face dimen-

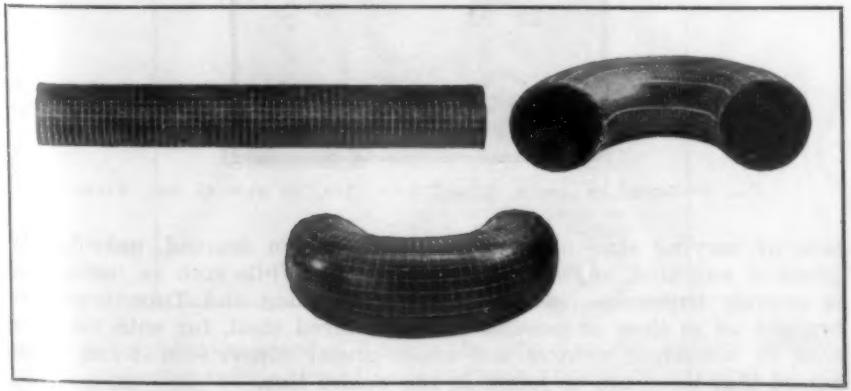


FIG. 7—TUBING MARKED IN EXACT SQUARES—AND SEVERAL VIEWS AFTER PROCESSING—SHOWING FLOW OF METAL

sion of a 1 in. 90 deg. turn is 1½ in.; that of a 2 in. is 3 in. and so on. The center to center dimension of a return fitting is three times the pipe diameter.

The ideal piping installation would be one of continuous true inside diameter and uniform wall thickness, or in other words a continuous

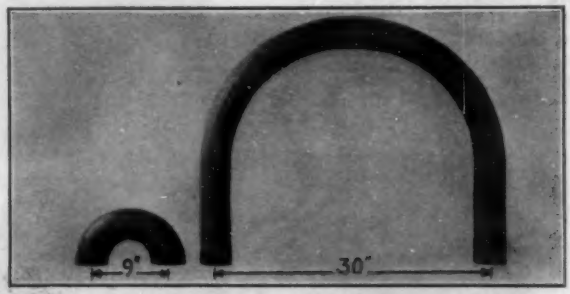


FIG. 8—A 3 IN., 180 DEG. TUBE-TURN COMPARED WITH A 3 IN. PIPE BEND MADE ON ADVISABLE RADIUS (5 TIMES PIPE DIAMETER)

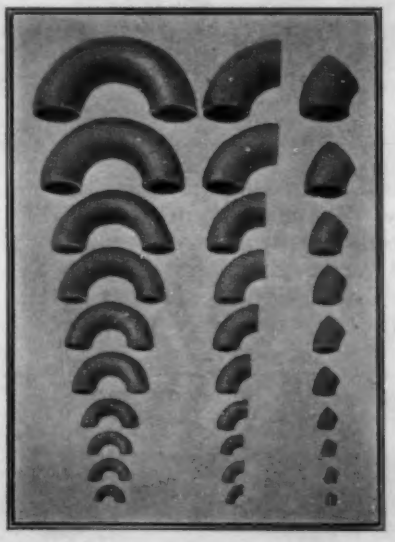


FIG. 9—SERIES OF STOCK TUBE-TURNS; 180, 90 AND 45 DEG. TYPES

tube of varying sizes as necessitated by design demand, unbroken by joints of any kind, anywhere in the system. While such an installation is entirely impossible, installations with welding and Tube-turns have brought us as close as possible to this desired ideal, for with the work done by competent welders and under proper supervision it can be assumed that there are no joints in the welded line.

One of the principal handicaps which the engineer specifying welding

has had to contend with was the lack of a proper fitting for making a change in direction in the completely welded job. Otherwise enthusiastic, in order to have an all welded system he has had to accept either long radius pipe bends or mitered elbows.

It has become quite common for engineers and others interested in piping problems to associate long radius bends with expansion ability.

They are used in practically all cases for this purpose because of the

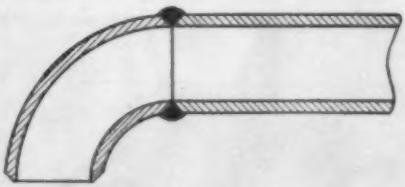


FIG. 10—THE TUBE-TURN BUTT-WELD JOINT — STRONG AS THE PIPE — AND LEAKPROOF. CONTRAST THIS WITH THE JOINT ILLUSTRATED BELOW

inability of cast fittings, either screwed or flanged, to absorb expansion stresses.

Having had in the past no other alternative than the use of the same material as the pipe in the line to take up these stresses, engineers have, therefore, of necessity, designed long radius pipe bends to the

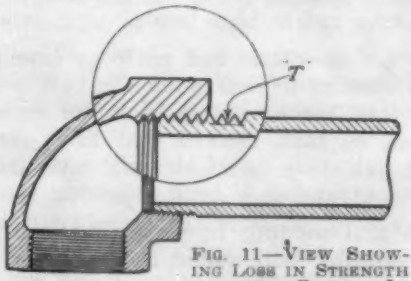


FIG. 11—VIEW SHOWING LOSS IN STRENGTH WHEN PIPE IS THREADED. THE EFFECTIVE STRENGTH OF THIS PIPE IS EQUAL ONLY TO THE WALL THICKNESS AT "T"

generally accepted standard of 5 times the pipe diameter because bending pipe to a shorter radius by ordinary methods entails deformation of the pipe and fracturing of the metal.

Pipe bends are extremely stiff; the inside wall tends to buckle in bending and the outside wall is stretched beyond a desirable point in fabricating and the question naturally arises as to the bend as such, having as much ability to take up expansion and contraction as the corresponding lengths of straight pipe and Tube-turns.

In the 1922 Transactions of the American Society of Mechanical Engineers there appears an article on the "Elasticity of Pipe Bends" by Messrs. Crocker and Sanford with an interesting discussion by J. A. Freiday giving valuable data on the advantages of expansion loops,

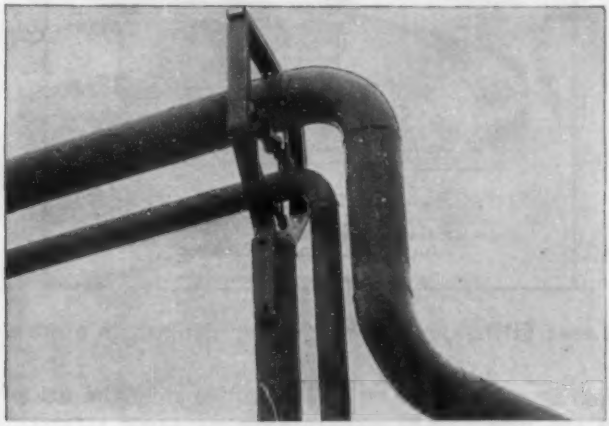


FIG. 12—TUBE-TURNS IN OVERHEAD PIPING REPLACING MITRE-WELD JOINTS

made up with straight lengths of pipe and fittings, over the usual type of loop made with long radius pipe bends.

Mr. Freiday and his associates had made an investigation some time previous to this discussion to determine the best method of providing expansion with the least reaction at the point of anchorage. The result of the investigation, he said, showed that loops corresponding to the expansion U bends, but made up of straight pipe and fittings produced less reaction than an expansion U bend requiring the same space.

Using tube-turns and welding in place of the fittings described by Mr. Freiday, the resulting loop should be far superior. The turns are stronger than the pipe, and fittings, either cast iron or cast steel, are prone to structural flaws especially when subjected to bending stresses as well as to high pressures and temperature strains.

In addition to the design advantages and saving in space afforded by this type of loop, there is a decided saving in cost. Long radius bends are awkward to handle. They are difficult to install when working in close quarters and few contractors are equipped to bend any but very small sizes either in their shops or in the field. Consequently bends are properly classed as "Specials" with all that the name implies, in

the way of high first cost, slow deliveries and increased cost of handling on the job.

The stock turns eliminate the necessity of having special bends made up in the fabricating shop and shipped long distances to the field. The contractor can make up his own expansion loops on the job avoiding sometimes serious delays and showing very decided savings in freight charges and handling and erection costs.

Until the introduction of tube-turns, the use of various types of mitered elbows in place of flanged or screwed fittings was fairly widespread, although designing engineers were not always inclined to favor their use. This type of fitting is inconsistent with sound engineering

FIG. 13—VIEW SHOWING ABSOLUTE UNIFORMITY OF WALL THICKNESS OF TUBE-TURNS

practice, because of the resulting high pressure drop. It makes a poor appearance, and its only recommendation was but no longer is, the necessity of its inclusion because of the lack of a proper fitting for the 100 per cent welded job. In short, it was and still is a makeshift, and because of resistance to flow, an efficient and satisfactory job can be secured only when the line is to operate under extremely low pressure conditions.

To obtain some data on the comparative resistances of the various types of elbows, an apparatus was composed at Rice Institute by J. H. Poung, Professor of Mechanical Engineering, by arranging in series, a steam turbine driven centrifugal pump, a Venturi meter and four different methods of making a 90 deg. Turn. The four 90 deg. elbows selected for comparison consisted of a standard cast iron screwed elbow and the three welded Turns shown in Figure 1. Only 3 in. standard pipe sizes were tested. In comparing the fittings for test, the welds were in all cases left just as they came from the welder. They were, however, good smooth welds internally with no bad drips or icicles projecting inward. It was realized at the outset that the unavoidable variations in the welds themselves would prevent fixing for the friction loss across a given type of welded turn a figure which would be both exact and general. But the fact that the results would not hold exactly true for the products

of every welder seemed no reason for failing to collect what data could be secured; and, since the aim of the test was to get an idea of actual field conditions, no special precautions to secure smooth internal surfaces were attempted. Variable quality in welding would certainly shift the friction loss curves of Figure 2 up or down. How much this shift might be was not determined.

The results of the tests are combined in the curves of Figure 2 which speak for themselves pretty well. They indicate for all types of turn the typical hydraulic condition of frictional loss increasing out of proportion of fluid velocity. One fact shown with striking clearness is that although the 90 deg. mitered fitting has only one welded joint between the piezometer connections as compared to the two welds needed in each of

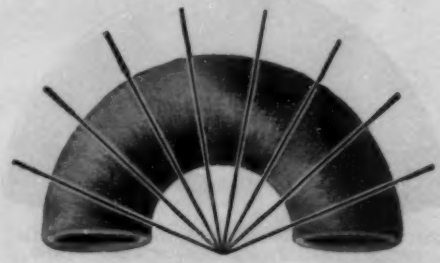


FIG. 14—THIS SKETCH SHOWS HOW TUBE-TURNS CAN BE CUT TO FORM PIPE TURNS OF ANY ANGLE. . . . EASILY INSTALLED, WITH ONLY TWO WELDS, EACH STRAIGHT ACROSS THE PIPE

the two other welded types—a fact which favors it both in cost and to a certain extent in fluid friction—its sharp angles much more than neutralize this latter advantage. So far as frictional resistance goes, it is by far the poorest of the turns examined. Professor Pound sums up his findings as follows:

"Power-bill savings can be applied against possible differences in first cost favoring the other turns against the cost of the installed Tube-turn. In case of a large installation, even small net savings per turn would total sums of real interest to the management."

While it has just been pointed out that tube-turns show an important saving in operating costs, the advantages shown in making these savings do not necessarily end here for an additional saving can be shown in initial expenditures as against the built up or mitered elbow, for one of the principal difficulties the welder has had to contend with was in fabricating the mitered elbow to obtain desired changes of direction in the piping system.

When the skill of the man doing the work is limited, the utmost advantage necessarily cannot be derived from the process. Some welders,

while adept at making their welds as strong and oftentimes stronger than the parent metal, have difficulty in fabricating special fittings cut to template.

It is not advisable, therefore, for a welder to undertake such exacting template work, unless he has had considerable previous training, since only the more highly skilled artisan can successfully carry on this class of work.

It seems reasonable to suppose that installing a stock tube-turn with two welds straight across the pipe is much the simpler job, saving extra cost in welder's time, lost motion and wasted material.

While a comparatively new product, they have been used extensively by many of the prominent engineers and by most of the large industrials

FIG. 15—CUTTING ODD ANGLE FROM 180 DEG. TUBE-TURN IN THE FIELD. NO DELAY AWAITING SPECIAL FITTING

and some of these companies have standardized on these fittings in changing direction on the welded piping job.

They are finding use in the oil industry, in power plants, in refrigeration work, in the process industries generally, and in ordinary steamfitting for both domestic and industrial heating systems.

With the increasing tendency of consulting engineers, utilities, industrials and piping contractors to rely on the welded piping job, the use of Tube-turns is becoming standard practice because of their adaptability for this modern method of pipe fabrication and they are taking their place with welding in giving every assurance of a more satisfactory piping design.

Another Oxy-Acetylene Welded River Crossing

By W. I. GASTON*

AS a part of its development program a large gas company in the Hudson River Valley recently planned to install a welded 8-in. high pressure line tying together three of its plants and also providing service for a number of communities that had not previously been supplied with gas. The line is designed for an ultimate pressure of 100 lb. per sq. in. but will carry about 45 lb. per sq. in. at first.

Two of these plants are about 15 miles apart on the same side of the river while the third plant is located on the opposite side of the river, the width at this point being slightly over a mile.

Construction of the shore line proceeded in accordance with standard practice. An interesting feature was the use of a gas wagon carrying an acetylene generator and cylinders of oxygen to supply the firing line gang of welders.

The river crossing, however, did involve special consideration and careful study was required in order to develop the best method for handling this part of the line. In determining the best procedure, the engineers of the gas company and of the pipe line contractor were assisted by the service engineers of the companies supplying gases and equipment for welding. It was decided to use 8-in. diameter extra heavy pipe for the river crossing.

The design developed for the crossing specified that the pipe was to be joined with a standard butt weld and that a reinforcing sleeve consisting of a 36-in. length of 9-in. standard pipe was to be placed over each joint and welded to the pipe at each end. The use of this sleeve also made it possible to test the three welds at each joint by applying pressure inside the sleeve. Specifications called for a test pressure of 350 lb. per square inch.

For use in making the crossing a railroad car ferry barge about 280 ft. long was obtained. With a barge of this length it was possible to schedule the various operations so as to insure continuous production at a maximum rate.

The procedure decided upon was, briefly, as follows:

A sufficient number of 40-ft. lengths of 8-in. extra heavy pipe were first double-lengthed to form 80-ft. sections. The sleeves were welded in place and, after testing, the pipe was coated with a protective material. Double-lengthing was done on the barge so that when this operation was completed the barge was ready to move into position for the crossing. The barge was of sufficient length so that three of these 80-ft. sections could be lined up along the center of the barge, giving

*Union Carbide Company.

FIG. 1 — UNLOADING PIPE ONTO BARGE

FIG. 2 — (LEFT) GENERAL VIEW OF DOUBLE-LENGTHING

FIG. 3 — (CIRCLE) SLEEVE ENDS SWAGED DURING WELDING

FIG. 4 — (BELOW) WELDING BUTT JOINT AND SLEEVE

three operating positions. In the first position nearest the front of the barge the butt weld was made; in the second position the sleeve was welded, and in the third position test pressure was applied to the sleeve. The barge was moved by means of a pile driver, using lines attached to anchors set in position along the line of crossing.

Before being allowed to work on the double-lengthing or river crossing, welders were required to pass tests for these operations. At all times the work of the welders was under the close observation of a competent inspector.

Double-Lengthing

In preparing the barge for the double-lengthing operation an inclined framework of heavy timbers was built so as to facilitate handling the pipe after it had been placed on the barge. The 40-ft. lengths of 8-in. extra heavy pipe were unloaded directly from the railroad cars onto this framework on the barge, as shown in Fig. 1. With the pipe placed as shown in Fig. 1, it was a relatively simple matter to line up two lengths of pipe and tack weld them in readiness for the butt welding operation. Before lining up the pipe the sleeve was slipped over the end of one of the lengths. These sleeves were 36-in. lengths of 9-in. standard pipe with the ends cut square. Each one had a small hole drilled and tapped in readiness for applying the pressure test. After tacking, the pipe was rolled toward the center of the barge, Fig. 2, where the butt weld was made, a single weld being used for this operation.

The pipe was then rolled to the next position, where the sleeve was moved into position over the butt weld, its correct location being determined by a mark on the pipe. Small cold chisels were used as wedges to obtain even spacing of the sleeve. Two operators work simultaneously in welding the sleeve. In order to facilitate welding, each end of the sleeve was swaged down to fit the pipe snugly as the weld progressed. That is, each welder would heat a short section of the end of the sleeve ahead of this weld and then hammer it down into contact with the pipe wall before proceeding with the weld. In Figs. 3 and 4 all of these operations are shown. The welder in the background is completing the butt weld. The two operators in the foreground are welding on the sleeve, the swaging operation being shown in Fig. 3.

The welded 80-ft. lengths were then ready for testing. Owing to the difficulty of obtaining a suitable compressor, it was decided to use compressed nitrogen for testing. Nitrogen was furnished in cylinders at pressure of 1800 lb. per sq. in. Being chemically inert, nitrogen is particularly well adapted for such high pressure tests. It is, of course, universally understood that oxygen must never be used under any circumstances for making pressure tests.

In making a test, a pressure gage, supplied with suitable fittings, was attached to the hole in the sleeve and connected by means of high pressure hose with a regulator attached to a cylinder of nitrogen. After a pressure of 350 lb. per sq. in. had been applied to the sleeve, the welds at each end of the sleeve were painted with soap suds and

carefully inspected, Fig. 5. Each sleeve was required to hold the test pressure of 350 lb. for 15 minutes without showing any drop on the gage.

After testing, the 80-ft. lengths were coated, as shown in Fig. 6.

A total of 72 double-lengths were prepared, as it was estimated that the crossing would require about 5700 ft. of pipe. As each joint

FIG. 5—TESTED AT 350 LB. PER SQ. IN.

required three welds, this meant that a total of 216 welds were made during the double-lengthing operation. The work was carried on continuously, the welders working in three 8-hour shifts. Double-lengthing was completed in about five days.

River Crossing

While the double-lengthing was in progress, preparations were made for handling the pipe during the river crossing. Heavy rollers were mounted along the center line of the barge for its entire length. These rollers were made from standard cable reels of the proper size, the core being filled with concrete to provide increased strength. In order to avoid damaging the protective coating on the pipe, the surface of each roller was covered with rope. The roller at the front end of the barge was mounted about 8 ft. above the deck of the barge and from this point the height of the supports or the rollers decreased regularly toward the rear end of the barge so that the pipe would be supported in an inclined position. This was done in order to minimize any tendency toward bending as the pipe was fed from the barge in making the crossing.

Two series of smaller rope-covered rollers were also provided, one on either side of the central line. These facilitated handling the

double lengths of pipe in bringing them into proper position on the main rollers.

A large automobile truck provided with a winch furnished the power necessary for moving the pipe into position.

Since it would be difficult to transfer materials to the barge after the crossing started, provision had to be made in advance for every possible contingency. Gases for welding were supplied from the gas

FIG. 6 (CIRCLE) PROTECTIVE COATING APPLIED HOT

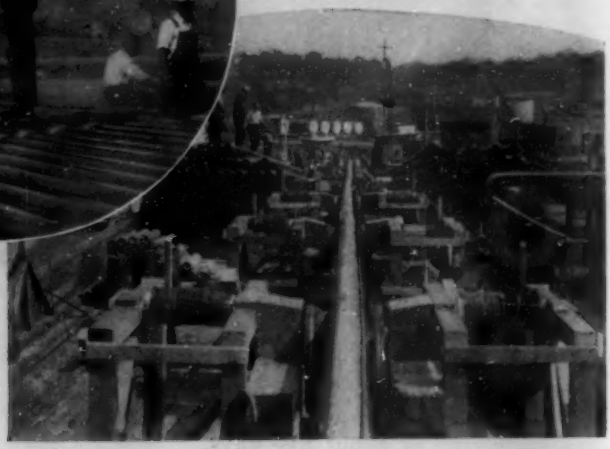


FIG. 7 — (BELOW) LOOKING AFT ALONG CENTER LINE OF BARGE

wagon that had been used very successfully in laying the line on shore. This wagon carried an acetylene generator and several oxygen cylinders. An ample supply of drums of calcium carbide and cylinders of oxygen was loaded on the barge. Cylinders of dissolved acetylene were also supplied for use while recharging the generator and also as an emergency measure. Cylinders of compressed nitrogen were also required for testing purposes.

A generator driven by gasoline motor supplied the current for the electric lights necessary to permit continuous operation. When the crossing started the men were required to work on alternate 8-hour shifts, but it was later found advisable to change this to alternate 12-hr. shifts. A cook house was installed on the barge so that meals could be served as required.

When all of the materials and supplies had been placed on the barge, it was moved into position to begin the crossing. Figs. 7 and 8 are

FIG. 8 — LOOKING FORWARD

FIG. 9 — (RIGHT) CLAMP LINES UP FOR TACK WELDING

FIG. 10 — (LEFT) BUTT WELD STARTED

views of the barge showing the general arrangement for handling the various operations.

The first step was to move three of the 80-ft. lengths of pipe on the rollers in the center of the barge. After a sleeve had been slipped over the end of one pipe at each joint, the joints were lined up and welded. The two sleeves toward the rear of the barge were welded. After testing the rearmost sleeve, the end of the line was drawn to shore and anchored.

Once the line had been started, a definite sequence of operations was established which continued throughout the entire crossing. Drawing the line to shore made room for another 80-ft. length at the front end

FIG. 11—WELDERS WERE QUALIFIED FOR POSITION WELDING

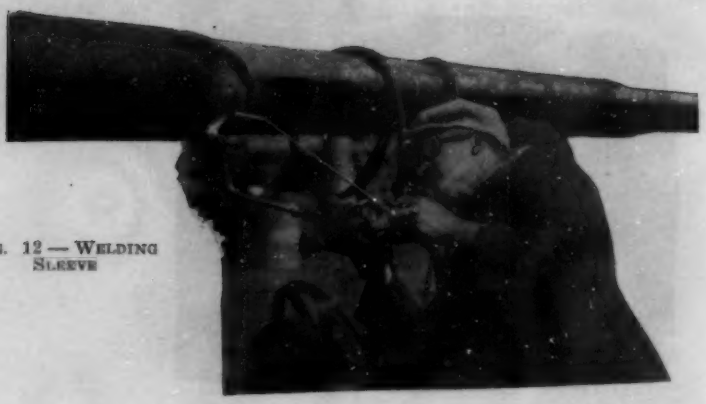


FIG. 12 — WELDING SLEEVE

of the line on the barge. By means of a cable attached to the winch on the automobile truck, a double-length was drawn from the pipe onto the small rollers set parallel to the central line. The length of pipe was then pulled on these rollers to the forward end of the barge, where it was moved into position on the main rollers. A supply of sleeves was maintained at this point of the barge and one of these was slipped over the end of the pipe before it was lined up. The type of pipe clamp used in lining up the pipe is shown in Fig. 9. Tack welds were made at four points, after which the clamp was removed and the butt weld completed, Figs. 10 and 11.

FIG. 13—UNDER TEST

While this was being done two welders were welding the sleeve over the joint at the second position toward the rear of the barge. At the third position a test was being applied to the completed joint, as is shown in Fig. 13.

The three operations of making the butt weld, welding the sleeve, and testing, all required about the same time for completion. When all three operations were finished the barge was removed 80 ft., another length of pipe placed in position and the cycle repeated.

A pile driver was used in moving the barge, anchors were placed in position ahead of the barge and lines were run from these over stanchions on the barge to the winches on the pile driver.

A cable from the truck winch on the barge was also attached to the line to facilitate moving along the rollers.

Pontoons were used to keep the line from sinking too rapidly to the river bed. These were welded steel oil storage drums which were at-

tached to the line at 50-ft. intervals, Fig. 14. Eight pontoons were used, the last two being moved forward as necessary, so that six were always in position supporting about 300 ft. of the line.

In spite of a severe storm the crossing was completed in 11 days. The line was simply laid on the bed of the river and subsequently divers using hydraulic jets cut a channel for the pipe at points where

FIG. 14—PONTOONS USED TO FLOAT LINE

there were abrupt changes in the river bed, particularly near the shores and at the sides of the dredged navigation channels.

The completed river crossing was tested by applying 150 lb. per sq. in. pressure, and requiring the line to hold this pressure for 12 hours without loss being shown on a recording gage attached to the line.

This crossing furnishes additional evidence of the value of procedure control in solving difficult welding problems.

An Investigation of Welded Connections Between Beams and Columns*

By Eugene H. Uhler† and Cyril D. Jensen†

1. Summary of Results.—(1) In every test where beams were welded to the columns, along the web of the I-beam, high stresses were developed in the weld under working loads.

(2) The indicated factors of safety for web-welded connections with working loads on the beams, ranges from 0.3 to 3.2. Even the higher of these values is not believed to be high enough for practice.

(3) The one test made on a short beam indicated that welding along the web of an I-beam may be satisfactory for very short beams.

(4) The few tests made on web-welded connections, where the lower flanges of the beams were in contact with the face of the columns, indicated an increased rigidity of the joint and at the same time reduced maximum stresses in the weld.

(5) Flange-welded connections gave nearly full rigidity of joint. Full negative moment should be used in the design of such joints.

2. Introduction.—Since the welded structure is a departure from the common riveted types, it seems that considerable study and laboratory work are necessary to discover the essential differences in design to insure safe structures. Designs of riveted connections of beams to columns are often made on the assumption of a hinged joint; or, in other words, the joint is designed to take end shear only. Practice has proved the worth of these riveted connections. However, the welded connection is so different from the riveted connection in a few fundamental respects that this assumption which is based upon riveting cannot be accepted as fitting the welded joint unless proved so by a suitable investigation.

The particular investigation reported herein was planned to test certain proposed designs and determine the stresses and moments created on the welded connection by known loads on the beam. At present, practice is too new to state definitely just what kinds of welded connections have been accepted. In Fig. 1 are shown some of the connections now in use. These joints resolve themselves into two groups, web-welded and flange-welded. Of these two groups, the web-welded connections seemed the more questionable and therefore were studied first. As shown in Fig 2, they consisted of a pair of fillet welds joining the web of the I-beam to the face of the columns. These connections will be referred to as web-welded joints. The other type of joint tested (Joints 6 and 7, Fig. 2) involved the welding of the flanges of the I-beam to the columns. These

*Paper to be presented at Annual Meeting, A. W. S., April, 1930.
†Assistant Professors of Civil Engineering, Lehigh University.

latter connections will be referred to as the flange-welded joints. The purpose of this paper, therefore, is to present the results of an investigation made on the strength, stiffness, and flexibility of certain welded connections between beams and columns.

Acknowledgment is made to the Bethlehem Steel Company for furnishing the beams and columns; also, to the Lehigh University for furnishing the services of a helper for a period of five weeks while the principal part of the investigation was under way. The authors are indebted to the following men: To Professors R. J. Fogg and M. O. Fuller

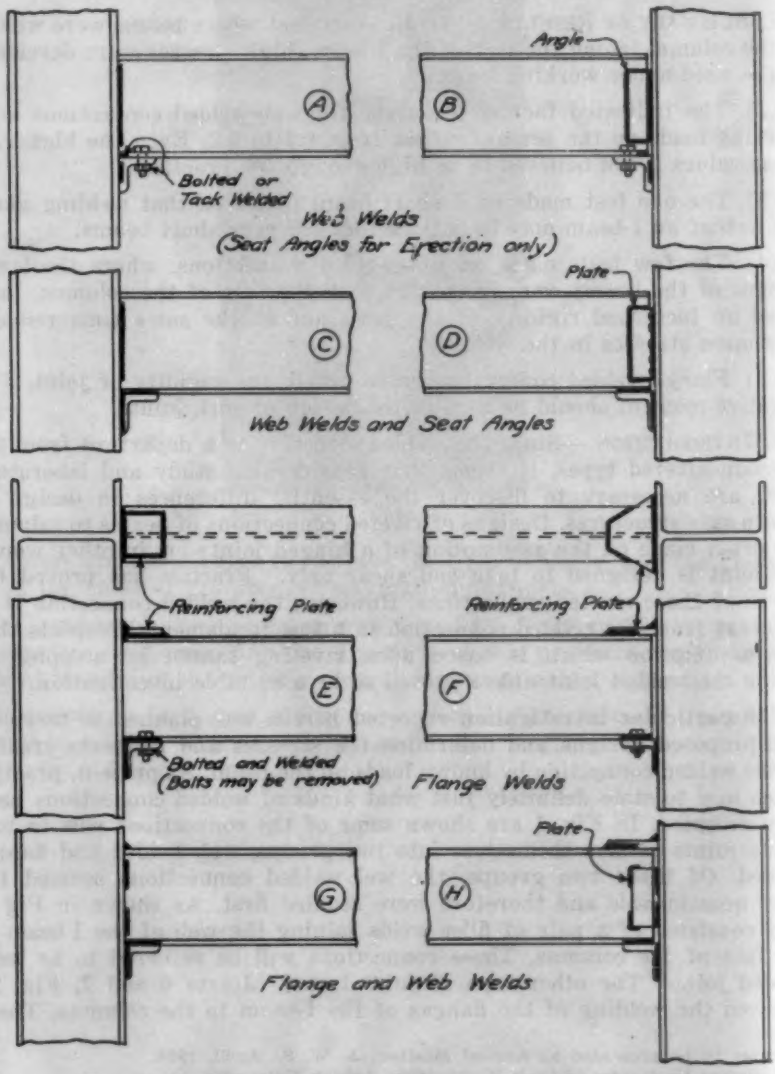


FIG. 1—BEAM-TO-COLUMN CONNECTIONS (PRESENT PRACTICE)

for advice and criticism; to C. L. Kreidler for efficient laboratory assistance; and to Professor W. A. Slater, Director of the Fritz Engineering Laboratory, for guidance and inspiration.

3. ANALYSIS.—The tests were made upon I-beams welded to columns. In the case of the cantilever beams, the resisting moment of the welds was the product of the applied vertical load, times its distance from the welds. In the case of a beam welded to a column at each end and loaded at the one-third points, the moment resisted by the weld is equal to the moment at the center minus the moment of the reaction about the third point. Therefore, to determine the end moment for the frames it was

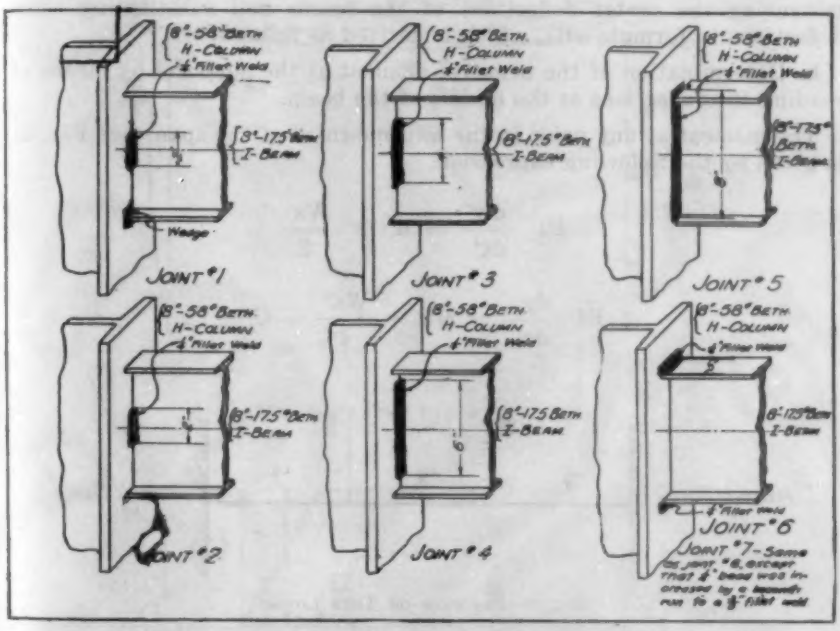


FIG. 2—DETAILS OF WELDED JOINTS

necessary to measure the resisting moment at the center. This method of measuring the resisting moments will be called the strain method, inasmuch as the measured strains formed the basis for computing the moments.

a. Determination of negative bending moments by the strain method.

$$M'' = \frac{SI}{C}$$

$$S = \epsilon E$$

Where M'' = moment at center of span
S = outermost fibre stress at center of span
ϵ = unit strain at point where stress S exists

It is known that for any beam with third-point loading

$$M' = M'' - \frac{WL}{6}$$

$$\therefore M' = \frac{\epsilon EI}{C} - \frac{WL}{6} \qquad (A)$$

Where M′ is the bending moment at the beam connections

A second method of determining the negative bending moments involved measuring the center deflections of the beams and substituting these deflections in formula (B), which is derived as follows:

b. Determination of the negative moment at the supports by means of reading the deflections at the middle of the beam.

The moment at any point in the left one-third of the span (see Fig. 3) is given by the following expression.

$$EI \frac{d^2y}{dx^2} = M' + \frac{Wx}{2}$$

$$EI \frac{dy}{dx} = M'x + \frac{Wx^2}{4} + C_1$$

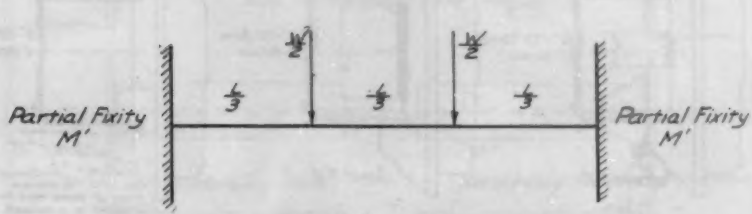


Fig. 3—Position of Test Loads

The moment in the middle third of beam is given by:

$$EI \frac{d^2y}{dx^2} = M' + \frac{WL}{6}$$

$$EI \frac{dy}{dx} = M'x + \frac{WLx}{6} + C_2$$

$$\frac{dy}{dx} = O, \text{ when } x = \frac{L}{2}$$

$$C_2 = -M' \frac{L}{2} - \frac{WL^2}{12}$$

Since the slopes are equal when $x = \frac{L}{3}$

$$\frac{M'L}{3} + \frac{WL^2}{36} + C_1 = \frac{M'L}{3} + \frac{WL^2}{18} - \frac{M'L}{2} - \frac{WL^2}{12}$$

$$C_1 = -\frac{M'L}{2} - \frac{WL^2}{18}$$

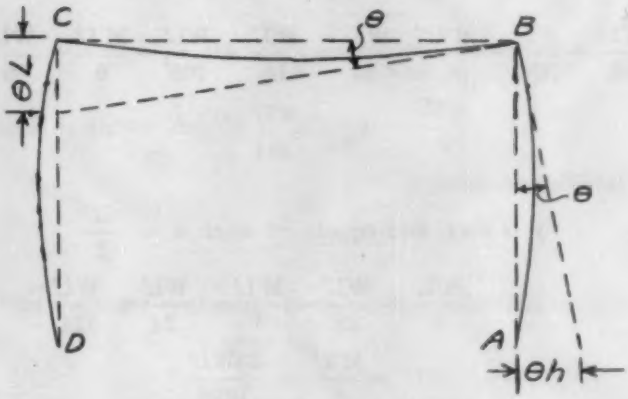


FIG. 4—ELASTIC CURVE OF FRAME

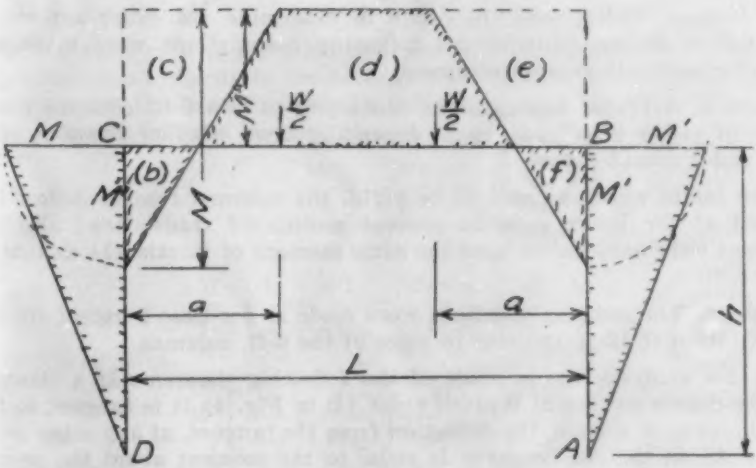


FIG. 5 BENDING MOMENT DIAGRAM OF FRAME

In the left one-third

$$EIy = \frac{M'x^2}{2} + \frac{Wx^3}{12} - \frac{M'Lx}{2} - \frac{WL^2x}{18} + C_3$$

$$y = 0 \text{ when } X = 0; \therefore C_3 = 0$$

In the middle one-third

$$EIy = \frac{M'x^2}{2} + \frac{WLx^2}{12} - \frac{M'Lx}{2} - \frac{WL^2x}{12} + C_4$$

The deflections "y" in the two parts of the beam are equal when $x = \frac{L}{3}$

$$\therefore \frac{M'L^2}{18} + \frac{WL^3}{324} - \frac{M'L^2}{6} - \frac{WL^3}{54} = \frac{M'L^2}{18} + \frac{WL^3}{108} - \frac{M'L^2}{6} - \frac{WL^3}{36} + C_4$$

$$C_4 = \frac{WL^3}{324}$$

In the middle one-third

$$y \text{ is max. and equals} -f \text{ when } x = \frac{L}{2}$$

$$-EIf = \frac{M'L^2}{8} + \frac{WL^3}{48} - \frac{M'L^2}{4} - \frac{WL^3}{24} + \frac{WL^3}{324}$$

$$EIf = \frac{M'L^2}{8} + \frac{23WL^3}{1296}$$

$$\text{From which } M' = \frac{8EIf}{L^2} - \frac{23WL}{162} \qquad (B)$$

c. Column Inclinations. In order to determine the exact amount of inclination of the columns, the following assumptions were made and then the inclinations were computed:

Case 2. A frame was assumed consisting of two 6-ft. columns and a beam of either 9 or 12-ft. span, depending upon whether Beam A or B was under consideration.

The joints were assumed to be rigid, the columns free to deflect but pinned at the lower ends to prevent motion of translation; also the columns were assumed to have the same moment of inertia (I) as that of the beam.

Case 3. The same assumptions were made as for Case 2 except for the substitution of 12-ft. columns in place of the 6-ft. columns.

In this analysis, use is made of the following theorem: If a straight line be drawn so that at a given point (B in Fig. 4) it is tangent to the elastic curve of a beam, the deflection from the tangent, at any other point (C or A) in the elastic curve is equal to the moment about the second point, of the area under the $\frac{M}{EI}$ curve.

(1) θ = slope at B (See Fig. 4)

θL = deflection at C, of the elastic curve, from the tangent

Moment of areas (c), (d), and (e) about C $= \dfrac{M''L^2}{2}$

Negative moment of areas (b), (c), (e), and (f) about C

$$= (M' + M'')a\frac{L}{2}$$

$$(2) \qquad \therefore \theta L = \frac{\dfrac{M''L^2}{2} - \dfrac{(M' + M'')aL}{2}}{EI_L}$$

But it is shown that $M' + M'' = M = \dfrac{Wa}{2}$

$$(3) \qquad \therefore \theta L = \frac{M''\dfrac{L^2}{2} - \dfrac{Wa}{2}\cdot\dfrac{aL}{2}}{EI_L}$$

From the same reasoning as above, the deflection at A is

$$\frac{\dfrac{M'h}{2}\cdot\dfrac{2h}{3}}{EI_h} \text{ that is}$$

$$(4) \qquad \theta h = \frac{M'h^2}{3EI_h}$$

If the joint at B is regarded as rigid, that is, maintaining a constant angle between tangents to the beam and the column at their intersection, regardless of the beam and column deflections, then the two slopes found from (3) and (4) can be equated.

$$(5) \qquad \theta = \frac{\dfrac{M''L}{2} - \dfrac{Wa}{2}\cdot\dfrac{a}{2}}{EI_L} = \frac{M'h}{3EI_h}$$

$$(6) \qquad \frac{Wa}{2I_L}\cdot\frac{L}{2} - \frac{M'L}{2I_L} - \frac{Wa}{2I_L}\cdot\frac{a}{2} = \frac{M'}{I_h}\cdot\frac{h}{3}$$

Grouping

$$(7) \qquad M'\left[\frac{h}{3I_h} + \frac{L}{2I_L}\right] = \frac{Wa}{2I_L}\left[\frac{L}{2} - \frac{a}{2}\right]$$

$$(8) \qquad \text{or } M' = \frac{Wa}{2}\left[\frac{L-a}{\dfrac{2h}{3I_h} + \dfrac{L}{I_L}}\right]\frac{1}{I_L}$$

$$(9) \qquad \text{or } M' = \frac{Wa}{2}\left[\frac{L-a}{\frac{2h}{3}\cdot\frac{I_L}{I_h}+L}\right]$$

for 1/3 point loading, $a = \frac{L}{3}$, also assume $I_L = I_h$

$$(10) \qquad M' = \frac{WL}{6}\left[\frac{\frac{2L}{3}}{\frac{2}{3}\cdot\frac{hL}{L}+L}\right] \text{ therefore}$$

$$(11) \qquad M' = \frac{WL}{6}\left[\frac{1}{\frac{h}{L}+\frac{3}{2}}\right]$$

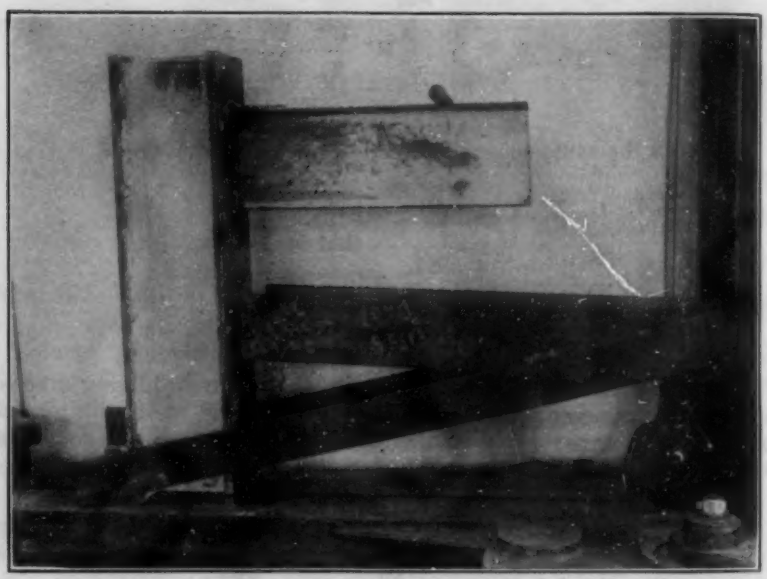


FIG. 6—CANTILEVER SPECIMEN FOR DETERMINING ULTIMATE LOADS

For any assumed rigid frame, of the type and with the loading shown in Fig. 15, the slope at the welded connections can be found by substitution in formula (5); whereas the negative moment, M′, may be found by substitution in formula (11).

4. CANTILEVER SPECIMENS.—The cantilever beams consisted of short 8-in. I-beams welded to 8-in. 58-lb. H-sections (see Fig. 6). The welder attempted to make the welded connections the same size as those in the corresponding frames described later so that a direct comparison could be made between the two sets of results.

5. FRAME SPECIMENS.—Two beams designated A (9-ft. span) and B (12-ft. span) served for making the complete series of tests. The beams were welded in turn between short 8-in. 58-lb. H-columns to form frames as shown in Fig. 7. The various welded connections investigated are shown in Fig. 2, and are numbered from 1 to 7. The test frames, then, will be designated A-1, A-2, B-1, B-2, etc., the letter referring to the beam, and the number to the joint.

6. PROPERTIES OF THE BEAMS.—The EI values of the beams, E being the modulus of elasticity of steel, and I being the moment of inertia of the beam, were determined experimentally for both Beams A and B. This

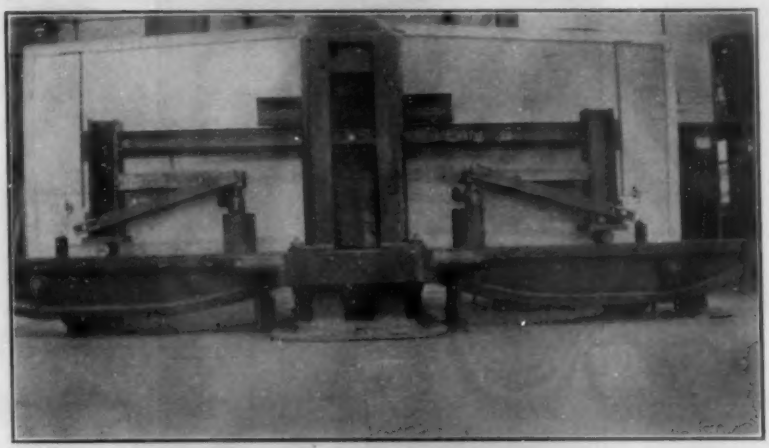


FIG. 7—GENERAL SET-UP OF FRAME

afforded more accurate results than if hand-book values had been used. Previous to being welded to the columns, each beam was set up in the testing apparatus as a simple beam loaded at the one-third points of the span. By measuring first the strain, and then the deflection at the center of the beam under known load, two EI values were determined for each beam from the relations $EI = \frac{WLc}{6s}$ and $EI = \frac{23WL^3}{1296f}$ respectively. These expressions are derived from the formulas for extreme fibre stress, and for center deflection of a simple beam loaded at the one-third points of the span. The EI values for Beam A averaged 1,678,000,000 lb.in.^{2}, both methods giving very nearly the same results. For Beam B the EI value by the strain method was 1,805,000,000 lb.in.^{2}, and by the deflection method, 1,650,000,000 lb.in.^{2}.

A plausible explanation of this wide variation in EI values for Beam B is that there may have been a flaw or irregularity in the beam outside the middle 10 inches. The EI value by the strain method is based entirely upon the resisting moment of the 10-in. middle portion of the beam;

whereas the EI value by the deflection method is based upon the resistance of all parts of the beam. A flaw or irregularity outside the middle 10 inches of beam would not change the 10-in. strain readings from those expected according to the accepted laws of stress variation but would change the maximum deflection of the beam. Therefore the EI values computed from these two sets of readings could not in such a case be expected to correspond. Using the two EI values for Beam B, each in its

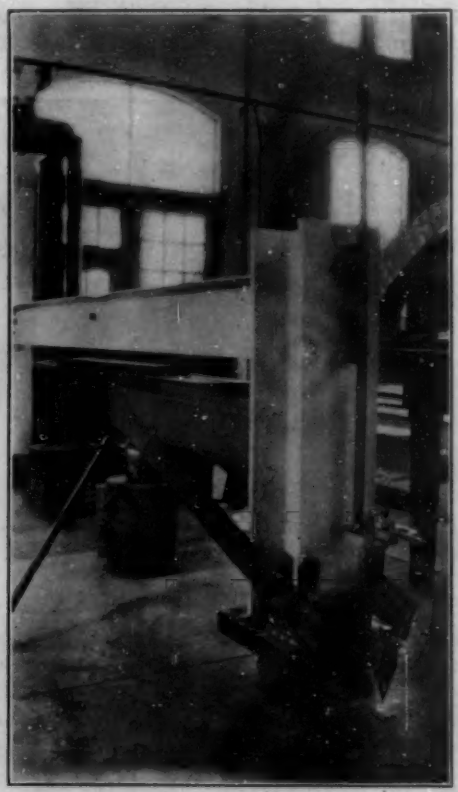


FIG. 8—ARRANGEMENT OF MICROMETERS

proper place, gave uniformly good results, which acted as a continual check on the original EI calculations.

7. THE WELDS.—Since it was desired to apply the results obtained to the welding of larger beams, the size of fillet weld was kept down to ¼-in., that is ¼ in. on each leg of the fillet, thus keeping the proportionate dimensions of fillet weld to I-beam somewhat like that actually used in practice. In every case the tolerance in size of fillet weld was less than 1/16 in. First 2 in. of weld (Joints 1 and 2, Fig. 2), were so placed on each side of the beam, that the weld was bisected by the horizontal center line of the beam. Then 2, 4, and 6 in. of welding were added respectively

to make Joints 3, 4 and 5. In every case the welds were kept symmetrical about the horizontal center line of the beam. The same procedure was followed in the flange-welded series. After Joint 6 (Fig. 2) had been tested with a ¼-in. fillet weld, a second layer of weld metal was added, increasing the size to a ⅜-in. fillet weld (Joint 7, Fig. 2).

8. METHODS OF TESTING.—The program of the frame tests was laid out in such a way as to test successively combinations of different welded

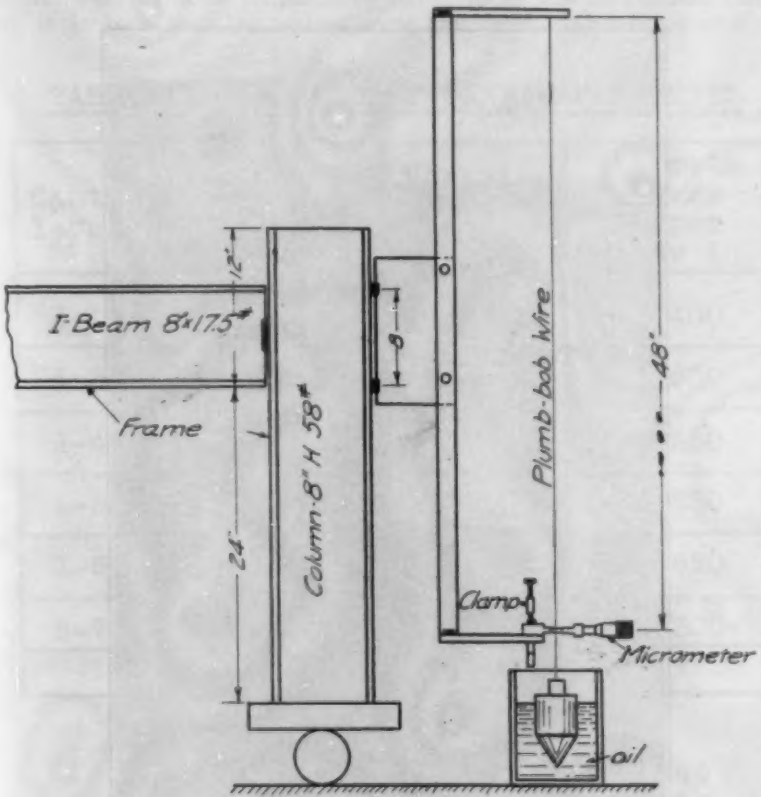


FIG. 9—APPARATUS FOR MEASURING INCLINATION OF COLUMNS

joints between the same beam and the same pair of columns. The only time that the welds were broken purposely, and the joints rewelded, was when changing from the web-welded series to the flange-welded series. With the properties of the beam carefully determined in advance, the comparison of the behavior of the different joints for stresses below the yield point was more reliable than would have been the case if different beams and different columns had been used in the cases compared. However, since it was important that the beams should not be over-stressed in succeeding tests, this procedure sacrificed the possibility of determin-

ing the maximum load and the factor of safety for each joint. The maximum moments of resistance for the different joints were, therefore, determined by tests on short cantilever beams. This gave the basis for an estimate of the factors of safety for the different types of joints.

a. *Cantilevers.* The point of application of the load for each cantilever test was so fixed that the same ratio of shearing stress to bending stress occurred in the welds of the cantilever as existed in the welds of the corresponding frame. Since the frame tests were made prior to the canti-

FIG. 10—DETAILS OF LUGS FOR READING WELD STRAINS

lever tests, the ratios were easily determined and the consequent arms for application of load computed.

The test of the cantilever beam was quickly and conveniently made. Figure 6 shows a cantilever specimen ready for the testing machine. Upon completion of each test, the ultimate bending moment was computed and entered with the other pertinent data in Table I.

b. *Frames.* It must be remembered that the purpose of the frame tests was to determine the bending stresses in the welds and the negative bend-

ing moments in the beams. It was important to avoid overloading in the frame tests, since too large a load would have caused permanent strains either in the welded connection or the beam, and would, therefore, have affected the remainder of the series. Considering Beams A and B in the frames as simple beams, the loads of 13,000 and 9,600 lb., respectively, will cause a computed maximum fibre stress of 16,000 lb. per sq. in. in the beams. Any restraint developed at the welded connections will raise the safe working beam loads. Since 10,000 lb. or less constituted the test load, it is seen that the beams were far from overstressed and that,

TABLE I - RESULTS OF CANTILEVER TESTS

Cantilever	Arm In.	Load In.	Moment in Inch-Lb.	Ultimate Stress lb.per Linear In.
A-1	8.9	14,400	128,000	12,400
A-3	6.1	10,900	66,500	12,500
A-4	12.6	7,850	99,000	8,250
A-5	17.6	9,500*	167,000*	7,830
B-6	27.4	11,500*	314,600*	7,870
B-7	28.2	16,000	451,200	11,300+
			Average -	9,770

* Average of two tests

\+ Not to be averaged with rest as it is a 3/8" Fillet weld.

therefore, the customary formulas for application below the yield point may safely be used in studying the test data.

Each frame was tested not only with the columns held vertical, but also with two predetermined inclinations of columns. These three conditions will be designated as Cases 1, 2, and 3. In the test frames, rather heavy columns, short enough to permit placing them in the testing machine, were used. However, it was desired to include in the investigation, a study of cases where long slender columns were caused to rotate and deflect due to the moment transmitted to them by the attached beam. An imaginary frame was assumed, employing higher and more slender col-

umns. The amounts of inclination of the columns at the connections were computed for the test loads (Formula 5 of the Analysis); and then the short columns used in the test were permitted to assume the computed inclination under the specified test load on the beam.

9. MEASUREMENTS.—For measuring the inclination of the columns, heavy mining plumb-bobs were tried and found very successful. These were hung from 48-in. brackets, which were tack-welded to the back face of the columns opposite the beam connection. The details are shown in Figures 8 and 9. Micrometers were fastened to the lower end of the 48-in. bracket to measure the inclination of the columns. Head-phones and an electric contact between plumb-bob wire and micrometer made it possible to take very accurate readings. The results showed that with the bobs immersed in oil, the micrometer could be successfully read to 0.001 in. A movement of 0.001 in. on the micrometer is equivalent to $\frac{0.001 \times 57.3 \times 3600}{48}$ or 4.3 seconds of arc. The best level bubbles available probably would not have given any greater accuracy.

Two ways of measuring the negative bending moments were used, the strain method and the deflection method. The strain method, as its name indicates, involves the measuring of the bending strains at the middle of the beam in the test frame, under known loads.

In Formula (A) $\left(M' = \frac{\epsilon EI}{c} - \frac{WL}{6}\right)$ of the Analysis, section 3, with the unit strain measured in the test, all terms are known except M′, which can, therefore, be computed. Strain readings on both edges of the flanges at the middle of the I-beam were taken with a 10-in. strain gage, reading to 0.0001 in.

The deflection method of determining the negative bending moment utilized formula (B) $\left(M' = \frac{8EIf}{L^2} - \frac{23WL}{162}\right)$ as derived in section 3. For measuring the center deflection of the beam, a small steel angle of the same length as the test beam was supported near its ends close to the web of the beam on pivot bearings carried by the web of the beam. An Ames gage reading directly to 0.001 in. was attached to the web of the test beam at the center of the span, its plunger bearing upon the steel angle. Deflection of the beam was thus indicated on the dial of the Ames gage. In this manner deflections were observed on both sides of the beam. The deflection apparatus may be seen rather indistinctly in Fig. 7, and the pivot support of the steel angle may be seen in Fig. 10.

For computing the maximum bending stresses in the welds proportionality between stress and vertical distance from mid-height of the beam was assumed. This leads to the equation $M' = \frac{Snd^2}{6}$ for the web-welded joints where M′ is the measured negative moment; S is the maximum intensity of the weld stress in lb. per linear inch of weld; n equals 2 for the two parallel fillets of a web-weld; and d is the length of the fillet.

This formula, however, does not apply for Joints 1 and 3a where wedges were used to put the lower flange of the beam in bearing. To compute the maximum intensity of the weld stresses for these joints, a graphic solution was used as shown in Fig. 11. For the flange-welded frames, the weld stress was computed from the formula $M' = Sdh$, where h, the only new term, is the lever arm between upper and lower flange welds. Each of the above formulas should be solved for S, the stress per linear inch of weld.

(C) Therefore, $S = \frac{6M'}{nd^2}$ for web-welded joints

(D) and $S = \frac{M'}{dh}$ for flange-welded joints.

In addition to computing the weld stresses due to the measured nega-

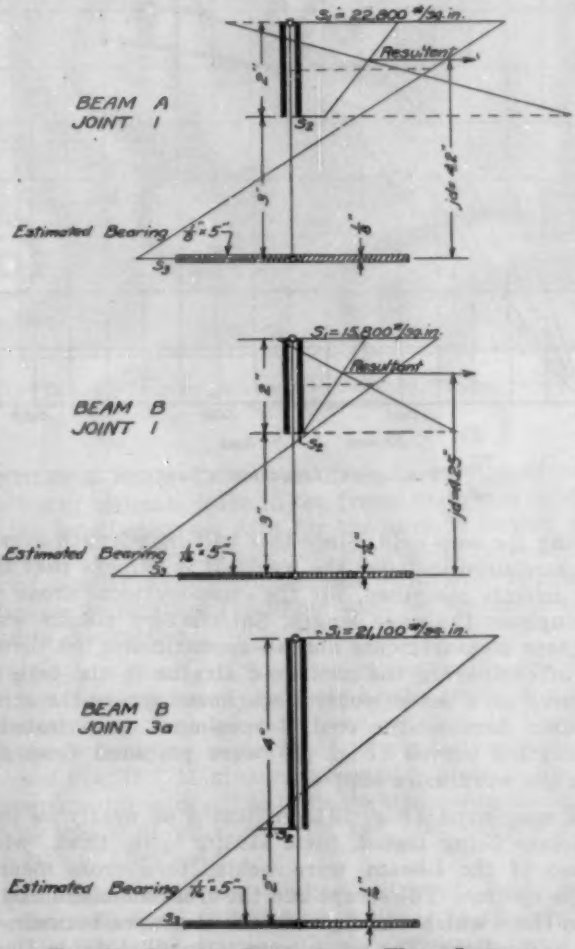


FIG. 11—GRAPHIC SOLUTION OF WELD STRESSES

tive moments, an attempt was made to measure directly, by means of a 2-in. strain-gage, the elongation in a gage length of ½ in. which, spanning from the column to the web of the I-beam, crossed the weld. Due to the concentration of the stresses in the welded joint and the rapid spreading out of these same stresses in the I-beam, a 2-in. gage length was considered too long to give satisfactory results. The device used for securing the equivalent of a ½-in. gage length is shown in the photograph, Fig. 10, Lug A was welded to the column and lug B to the web of the I-beam at a distance of ½ in. from the face of the column. The relative movement between the holes in these lugs was measured with the 2-in. strain gage, but this represents the elongation in the ½-in. gage

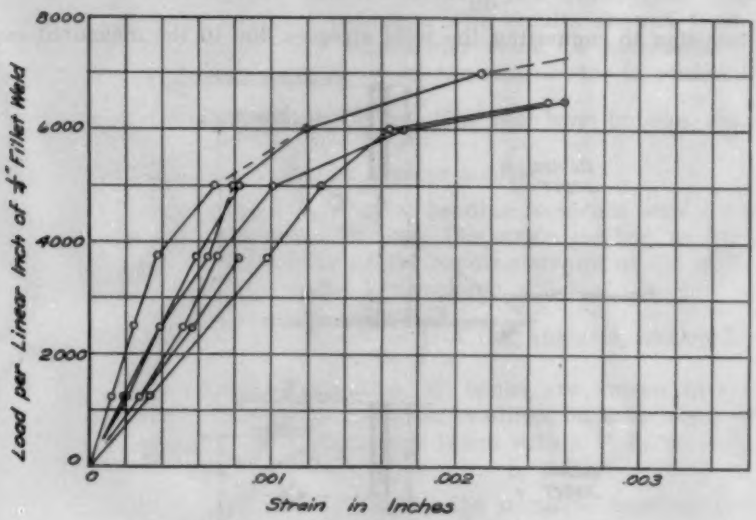


FIG. 12—CALIBRATION CURVE

length spanning the web-weld. Since this half-inch length over which the strains were measured included the welds, it is evident that the stresses could not be directly computed, for the cross-sectional areas were never constant throughout the gage length. Satisfactory results were not secured from these measurements and an approximate, but direct, method was evolved of comparing the measured strains in the tests to similar strains measured on a set of control specimens where the stresses were definitely known, because the control specimens were tested in direct tension. Calibration curves (Fig. 12) were prepared from the control tests to make the work more simple.

The control specimens (Fig. 13) duplicated as nearly as possible the web-welded joints being tested. Steel straps ¼-in. thick, which represented the web of the I-beam, were welded to a cross member which represented the column. The straps and the cross members had a space of ⅛-in. between them which was equal to the clearance between the end of the test beam and column. The welds were ¼-in. fillet welds, the same size

as were used on the frames and cantilevers tested. Strains were read on the control specimens under known loads using a 2-in. Tinius Olsen strain gage. The resulting strain-load curves for several such control tests are shown in the calibration curve in Fig. 12. It was thought more satisfactory to use this composite curve of the control tests for reading directly, even though only approximately, the stresses obtained in the frame tests than to resort to a still more uncertain computation based upon the product of the modulus of elasticity and the strain measured in the beam across the weld.

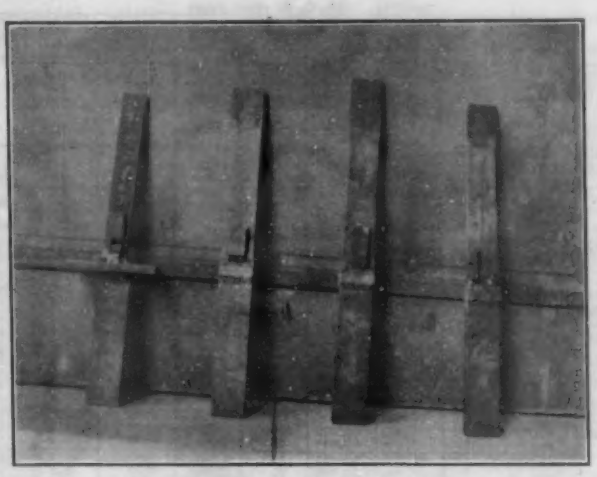


FIG. 13—TENSION SPECIMENS USED TO CALIBRATE 2-IN. STRAIN READINGS

10. CANTILEVER BEAM TEST RESULTS.—As mentioned previously, the determination of ultimate loads in the frame tests was sacrificed in order to secure the bending stress data for the various welded connections all on the same pair of beams. Therefore, in order to determine the ultimate strengths of similar welded connections, cantilever beams were welded to the columns, duplicating the joints used in the frame tests, and were then tested to destruction.

Table I gives the results of the tests on the cantilevers. For simplicity, each specimen was numbered the same as the frame it represented. The selected moment-arm for each cantilever, as given in the second column, was the distance from the connection to the inflection point in each frame specimen (see Fig. 15). Maximum moments as given in the fourth column are the experimental values. The ultimate stresses in the last column were computed by means of equations (C) and (D) and are given in pounds per linear inch of fillet weld.

Straight tension tests on similar pairs of fillet welds developed strengths from 8000 to 10,000 lb. per linear in. Referring to the last column of the table, the average ultimate stress is found to be 9770 lb.

per linear in. per ¼-in. fillet. The agreement in ultimate strengths as found by the two methods is found to be very good.

An interesting comparison is afforded between cantilevers A-4 (6 in. of web-welding) and B-6 (flange-welding—5 in. across the top and bottom flanges). Whereas A-4 has a total of 12 in. of fillet weld at the connection and B-6 has 10 in., the maximum resisting moments are 99,000 in.lb. for A-4, and 314,600 in.lb. for B-6. This shows an immense gain in strength due to the different placement of the welds

TABLE II. RESULTS OF FRAME TESTS

Frame	Span	Joint	Column Inclination	Test Load	Test Results: Negative Moments Measured	Test Results: Negative Moments Computed as	Test Results: Per Cent Rigidity	Weld Stresses: Computed	Weld Stresses: Measured	Estimated Factor of Safety when designed as: Simple Beam	Restrained Beam ***	Fixed Beam
A-1	9	2"web-weld	Case 1	10000	44,500	120,000	37	4000	5400	1.9	1.5	1.3
		(wedges)	Case 2	"	22,400	83,100	27	2010	4500	3.7	3.0	2.5
			Case 3	"	17,500	63,500	28	1720	4400	4.4	3.5	3.0
A-2	9	2"web-weld	Case 1	2000	2,800	24,000	12)					
			Case 2	"	3,600	16,600	22)	2850		0.5	0.4	0.3
			Case 3	"	5,000	12,700	39)					
A-3	9	4"web-weld	Case 1	7500	23,000	30,000	76	4300	7000	1.3	1.1	0.9
			Case 2	"	19,000	82,300	29	3380	6330	1.7	1.3	1.1
			Case 3	"	15,000	47,600	32	2800	6100	2.0	1.6	1.4
A-4	9	6"web-weld	Case 1	10000	63,000	120,000	52	5250	---	1.4	0.9	0.9
			Case 2	"	42,000	83,100	51	3500	6600	2.1	1.4	1.4
			Case 3	"	32,000	63,500	50	2670	6800	2.9	1.9	1.9
A-5	9	9"web-weld	Case 1	"	88,000	120,000	73	4130	6500	1.9	0.9	1.2
			Case 2	"	66,000	83,100	79	3100	5800	2.4	1.2	1.6
			Case 3	"	50,000	63,500	79	2350	4400	3.2	1.6	2.2
B-1	12	2"web-weld	Case 1	5000	25,900	90,000	32	4940	6700	1.0	0.5	0.7
		(wedges)	Case 2	"	19,700	60,000	33	3770	6100	1.3	1.1	0.9
			Case 3	"	11,000	48,000	23	2110	4000	2.4	1.9	1.6
B-2	12	2"web-weld	Case 1	2000	2,900	32,000	9	2170	---	0.9	0.9	0.6
			Case 2	"	2,200	24,000	9	1650		1.2	1.1	0.8
			Case 3	"	2,200	19,200	11	1650		1.2	1.1	0.9
B-3	12	4"web-weld	Case 1	5000	25,200	80,000	32	4730	7300	1.1	0.8	0.7
			Case 2	"	19,600	60,000	33	3680	6400	1.4	1.1	0.9
			Case 3	"	15,100	48,000	31	2830	5300	1.8	1.4	1.2
B-3a	12	(wedges)	Case 1	"	39,500	80,000	49	3780	7000	1.4	0.9	0.9
B-4	12	6"web-weld	Case 1	"	48,500	80,000	61	4050	6400	1.3	0.8	0.9
			Case 2	"	38,200	60,000	64	3180	6100	1.5	1.1	1.1
			Case 3	"	29,100	48,000	59	2340	3800	2.2	1.3	1.5
B-5	12	9"web-weld	Case 1	"	61,400	90,000	77	2590	6500	1.9	0.9	1.2
			Case 2	"	48,700	60,000	81	2280	4200	2.2	1.2	1.5
			Case 3	"	40,500	48,000	94	1900	3700	2.7	1.4	1.8
B-6	12	1/4 in. flange weld	Case 1	10000	137,000	160,000	86	3430		3.0	1.7	2.0
			Case 2	"	107,000	120,000	89	2680		3.8	2.2	2.5
			Case 3	"	93,000	96,000	97	2070		4.9	2.8	3.3
B-7	12	3/8 in. flange weld	Case 1	"	141,000	160,000	88	3525		3.3	2.0	2.2
			Case 2	"	109,300	120,000	91	2725		4.3	2.6	2.9
			Case 3	"	91,000	96,000	95	2275		5.2	3.2	3.5

* Case 1.Columns held vertical
Case 2.Columns inclined for a frame with 6ft.columns.
Case 3.Columns inclined for a frame with 12ft. columns
** Computed on basis of a rigid joint.
***For the restraint found in each welded joint.

11. FRAME TEST RESULTS.—The results of the frame tests are summarized in Table II. Each of the measured negative bending moments listed is the average of two determinations, one by the strain, and one by the deflection method. Figure 14 shows the agreement between these two methods for Case 1 where the columns were held vertical. The curves for Cases 2 and 3 show a similar agreement. The next column in the table, Computed Negative Moments, was computed on the assumption of rigid joints. To differentiate between the terms expressing rigidity and fixity, a rigid joint has been defined as one maintaining a constant angle of intersection between axes of column and beam, regardless of the amount of rotation of the joint; whereas a fixed joint, will be defined as a rigid joint in which there is no rotation due to the bending moment. Therefore, Case 1, where the columns were held vertical, corresponds to a fixed condition if the welds are assumed to be rigid. Cases 2 and 3, on this same

assumption, fall into the class of rigid joints. For all three cases, formula (11) of the Analysis is used for computing the negative moments based on rigid joints. (Note, for Case 1, h is zero.)

In order to visualize the stiffening effects of the various welded joints more clearly, moment diagrams have been prepared (see Figs. 15 and 16). In each moment diagram the top moment curve is that for a simple beam; whereas the bottom moment curve is that computed on the basis of rigid joints on frames with columns held vertical or on frames with long slender columns as assumed in the analysis. The fact that the moment curves from the test results fell between the two extremes showed that a so-called partial rigidity of the welded connections was obtained.

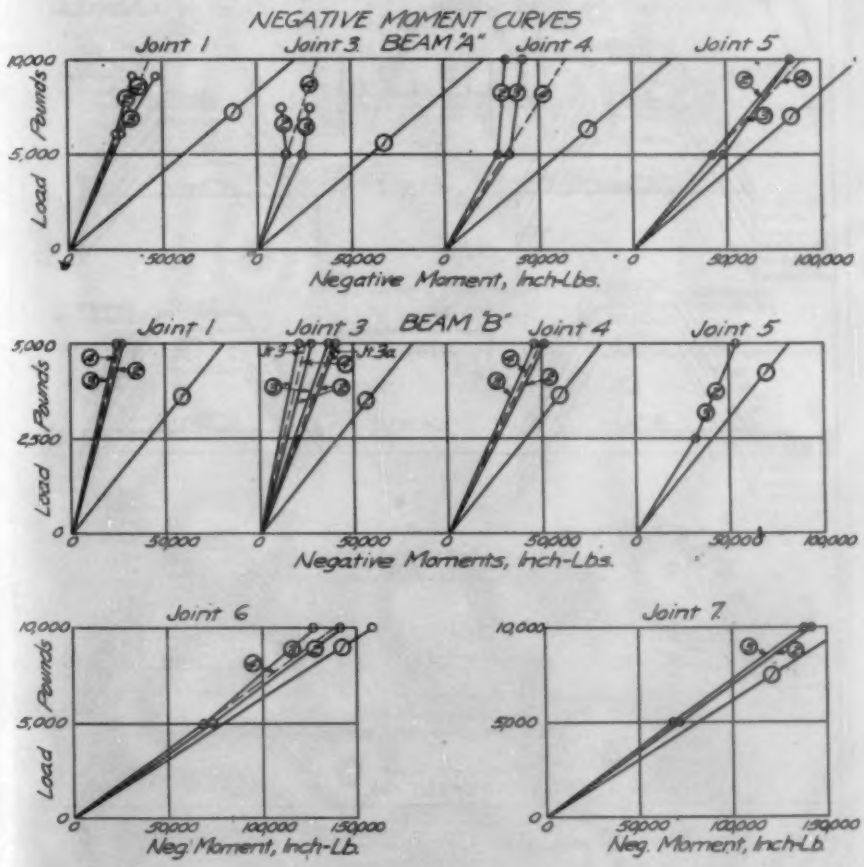


FIG. 14—NEGATIVE MOMENT CURVES (CASE 1)

The percentages of rigidity as listed in Table II are the ratios of measured to computed negative moments, multiplied by 100 to reduce to percentages. A study of the percentages indicates that for the weaker connections, especially Joint 2, it was difficult to secure consistent results for the measured negative moments. Otherwise it appears that generally the percentage of rigidity of a particular joint was nearly a constant* regardless of whether or not the columns were held vertical or were inclined. Furthermore, the percentages of rigidity for Beam A are in fair agreement with the percentages for the same connections for Beam B, indicating that length of beam may have little if any bearing on the rigidity of a welded joint.

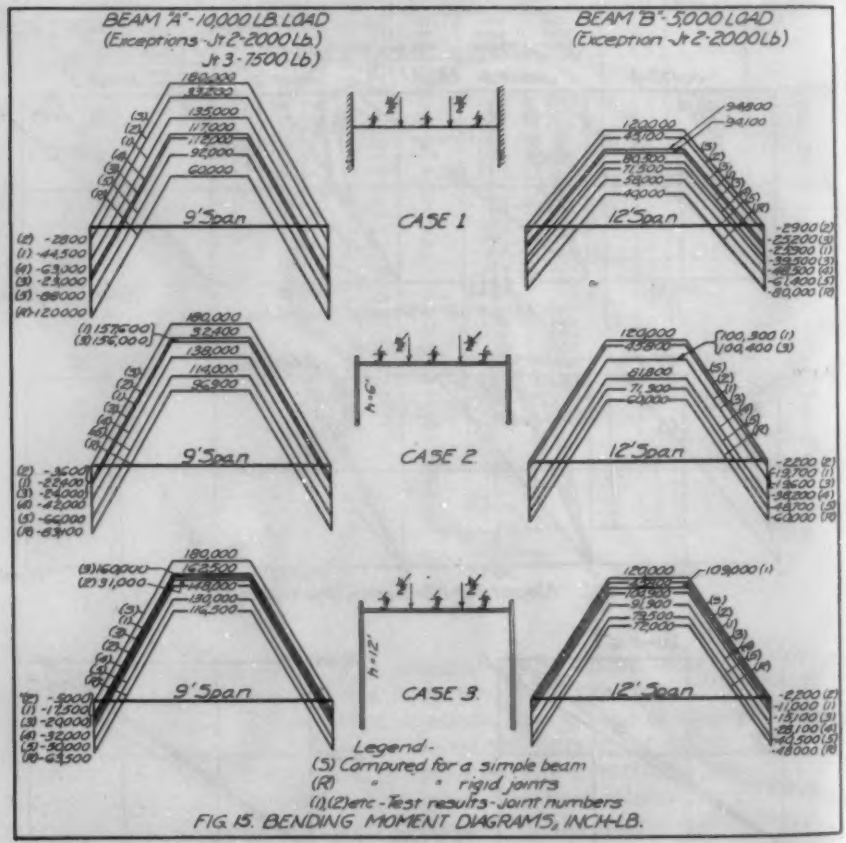


FIG. 15. BENDING MOMENT DIAGRAMS, INCH-LB.

FIG. 15—BENDING MOMENT DIAGRAMS FOR WEB WELDS

*The more rigid joints, A-5, B-5, B-6, and B-7 indicated an upward trend of the percentage of rigidity as the column inclinations were increased; but the increase seems small enough to neglect. In the less rigid joints this upward trend seems to compensate generally for the reduction of measured negative moment, which is caused by permitting the columns to incline more than they naturally would in the assumed frames with welded joints low in rigidity.

It is to be noted in Figure 2 that Joints 1 and 2 differed only in that wedges in Joint 1 put the lower flanges of the beam in bearing against the columns. The effect of the wedges was to stiffen the beam as noted in the percentages of rigidity of A-1 and B-1 compared with the percentages

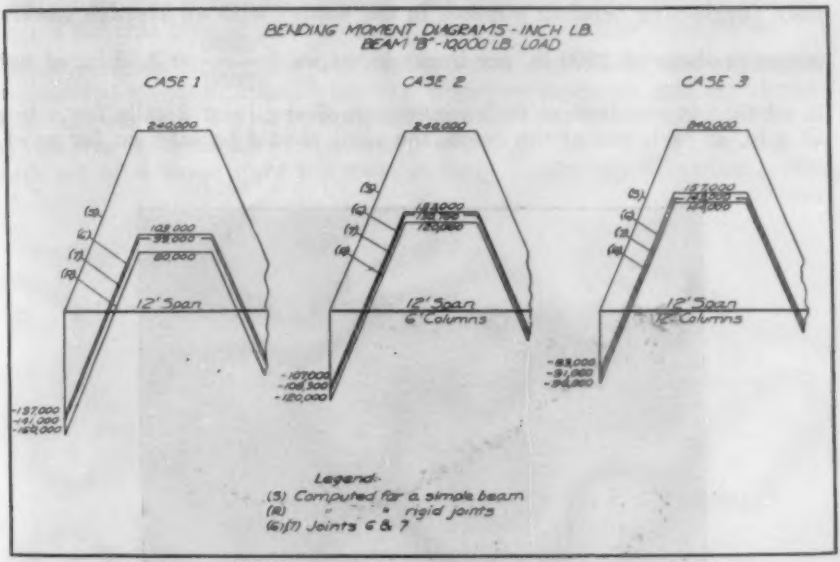


FIG. 16—BENDING MOMENT DIAGRAMS FOR FLANGE WELDS

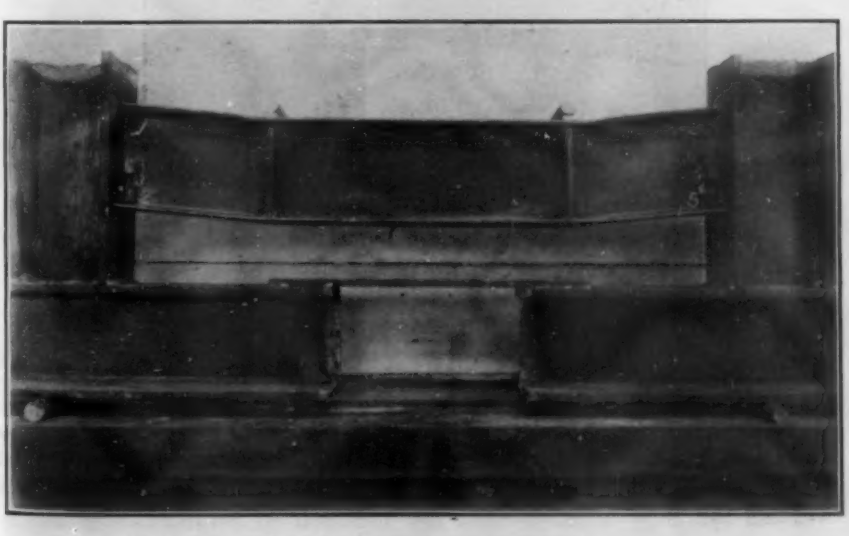


FIG. 17—SHORT BEAM SPECIMEN

for A-2 and B-2. This stiffening effect was verified again in the test of Frame B-3a as compared with Frame B-3, Case 1.

Joint 2 is undeniably about the weakest looking web-welded joint which the most daring of designers might consider for practical use. However, it does have some favorable points. Designed on the basis of end shear only (neglecting bending stresses in the welds) with an average working stress in shear of 2000 lb. per linear in. of weld, $\frac{6500}{2000}$ or 3.25 in. of weld is all that is required at each connection. Since Joint 2 calls for a total of 4 in. at each end of the beam, the joint should be safe, so far as end

FIG. 18—WELD FRACTURE—SHORT BEAM SPECIMEN

shear is concerned. However, as noted in Table II the bending stresses were far from negligible. In fact, the estimated factors of safety, based on safe working loads on the beams, fell below unity for Case 1 of Frames A-2 and B-2.

The data of the other web-welded joints in the order, 3, 4, and 5, show

successively increasing rigidity, Joint 5 having approximately 80 per cent of perfect rigidity. These percentages of rigidity are of interest since. so far as is known, there is no analytical method for computing the rigidity of the joints.

Frames B-6 and B-7 were the only ones having flange-welded connections. As soon as it became apparent that practically full rigidity seemed assured for this type of joint no more frames of this type were tested.

Weld stresses under the test loads were determined as explained previously by computation from the negative moments and by direct measurements with a 2-in. strain gage. A wide variation is noted between the two sets of stresses (see Columns 9 and 10 of Table II). To determine which set of stresses gave the more reliable results the following study

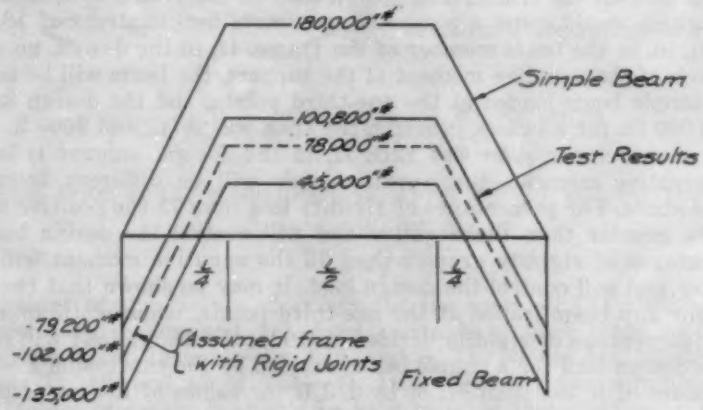


FIG. 19—MOMENT DIAGRAMS SHORT BEAM TEST

was made: Using 8000-10,000 lb. per linear in. as the ultimate strength of the welds, the ultimate loads in the frame tests can be estimated by the proportion: ultimate load is to test load as ultimate weld stress is to weld stress at test loads. Using first the computed and then the measured weld stresses for test loads, the following table has been prepared using as ultimate weld stresses 8000 and 10,000 lb. per linear in.

TABLE III—ESTIMATED ULTIMATE LOADS FOR THE FRAMES

Frame	Estimated Ultimate Load on Frame According to Computed Weld Stresses	Measured Weld Stresses	Cantilever Test
A-1	20,000–25,000	14,800–18,300	28,800
A-3	13,900–17,400	8,600–10,700	21,800
A-4	15,200–19,100		15,700
A-5	19,300–24,200	12,300–15,400	19,000

The last column in the table is the estimated ultimate load for the frames as determined by doubling the ultimate loads obtained for the corresponding cantilevers. Since the computed stresses seem to be in

better agreement with the results of the cantilever tests of Table I, more weight apparently should be given to the computed stresses than the measured ones.

The web-welded connections appear to be badly overstressed, when it is considered that the safe working stress often used in practice is 2000 lb. per linear in. of ¼-in. fillet weld. Moreover, the test loads used in the investigation were less than the allowable working loads for the beams. Therefore, an extended study is hardly necessary to reach the conclusion that if full working beam loads had been applied some of the connections might have failed, and certainly all of them would have been badly strained.

12. FACTOR OF SAFETY.—In this discussion the term Factor of Safety is the ratio of the load, which would cause failure of the weld, to the design load on the frame. The design load on the frame is defined as the load which would cause a computed maximum tensile stress of 16,000 lb. per sq. in. in the beam member of the frame. If, in the design, no account is taken of the negative moment at the support, the beam will be designed as a simple beam loaded at the one-third points, and the design load will be 13,000 lb. for all cases in which the span was 9 ft., and 9600 lb. for all cases in which the span was 12ft. If, in the design, account is taken of the negative moment, the working loads will be different from those stated above. For percentages of rigidity less than 75 the positive moment will be greater than the negative and will control the design load. For percentages of rigidity greater than 75 the negative moment will be the greater and will control the design load. It may be shown that the design load for any beam loaded at the one-third points, whose ratio of rigidity is R (percentage of rigidity divided by 100), the design load will be equal to the design load for a simple beam, divided by the expression $1 - 2/3\ R$ for values of R less than ¾, or by $2/3\ R$ for values of R greater than ¾.

Since the frames were not tested to failure the maximum load has been estimated on the assumption that the stress in the weld would be proportional to the load up to the point of failure of the weld. With these assumptions expressions for the estimated factor of safety have been derived. The notation used in this derivation is as follows:

W_M = maximum load

W_T = test load

W_D = design load for simple beam

9770 = computed bending stress in the weld at the maximum load (average stress from Table I)

S_T = computed bending stress in the weld at the test load (see Table II Column 9)

S_D = computed bending stress in the weld at the design load

F = factor of safety

Since stress is assumed to be proportional to load,

$$(1) \qquad \frac{W_M}{W_D} = \frac{9770}{S_D}$$

(2) $$\frac{S_D}{S_T} = \frac{W_D}{W_T}$$

(3) $$\therefore S_D = S_T \frac{W_D}{W_T}$$

(4) $$F = \frac{W_M}{W_D} = \frac{9770}{S_T \frac{W_D}{W_T}} = \frac{W_T}{W_D} \frac{9770}{S_T} =$$ factor of safety when the beam is designed as a simple beam. For a restrained beam the design load is

$$\frac{W_D}{1 - \frac{2}{3} R}$$

for $R < \frac{3}{4}$ or $\frac{W_D}{\frac{2}{3}}$ for $R > \frac{3}{4}$. Therefore a restrained beam

(5) $$F = (1 - \frac{2}{3} R) \frac{W_T}{W_D} \frac{9770}{S_T} \text{ or } \frac{2}{3} R \frac{W_T}{W_D} \frac{9770}{S_T}$$

A fixed beam is the special case in which $R = 1$ and for this case,

(6) $$F = \frac{2W_T}{3W_D} \frac{9770}{S_T}$$

Estimates of the factors of safety for the cases of (a) simple beam, (b) restrained beam, and (c) fixed beam, have been made using the expressions given in equations 4, 5, and 6. The values are tabulated in Columns 11, 12, and 13 of Table II. In considering the significance of these factors of safety it should be kept in mind that the design load is the load which will cause a computed fibre stress of 16,000 lb. per sq. in. in the beam, without reference to the stresses in the welds, whereas the maximum load is in all cases the load which would cause a stress of 9770 lb. per linear in. in the weld, assuming that at the maximum load the negative moment bears the same relation to the simple beam moment, $\frac{Wl}{6}$, that it did at the test load. In view of the fact that yielding of the welded joint under high stresses would cause some redistribution of moments it is likely that the factor of safety would be somewhat greater than the estimated values given, if the maximum loads had been determined by testing the frames to failure.

The factors of safety indicate all of the web-welded joints to be dangerous. On the other hand, the factors of safety for the flange-welded connections, while not high enough to satisfy present practice, are much more favorable than those found for the web-welded connections. It has been shown in other investigations that by means of reinforcing plates, that flange-welded connections can be designed to be as strong as, or stronger than the beam connected.

13. SHORT BEAM SPECIMEN.—The preceding investigation was made

on beams of medium length compared to their sections. Before arriving at too definite conclusions regarding the web-welding of beam-to-column connections it may be well to review briefly a test, made in preparation for the investigation, in which a web-welded joint was tested on a very short beam.

An 8-in., 17.5-lb. Bethlehem I-beam 4-ft. long was web-welded between two short stiff columns. The columns were heavy 8-in. H-sections and were firmly welded to similar sections serving as bases. Figure 17 shows the beam after failure. The lugs which appear in the photograph at the web connections to the short columns were for gage readings and had no influence on the strength of the beam or joints. Tie-rods between the bases prevented any possible spreading and therefore helped materially in holding the columns vertical. As shown in Fig. 18, the web of the I-beam was welded to the face of the columns using a ¼-in. fillet weld. For this particular span of 4 ft. the safe working load for quarter point loading (used only in this test) was found to total 38,400 lb. Designing the welded connections for end shear only, on the basis of 2000 lb. per linear in. as a safe working weld stress, 10 in. of welding should be needed on each end of the beam. Actually 12 in. were supplied, 6 in. being placed on each side of the web.

The apparatus for measuring the inclinations of the columns had not yet been developed. Therefore, it was not known just how much the columns inclined during test. The inclination could not have been much, however, due to the method of fastening the columns to the bases, and also due to the tie-rods; and yet it is fairly certain that some inclinations took place. The strain method was the only way used to measure the moments developed. Since the determination of the negative moment by the strain method involves the subtracting of the actual moment at the middle of the beam from the total moment, the resultant negative moment is not dependent upon a knowledge of the exact inclination of the columns. The results are presented for a load of 30,000 lb. in the moment diagrams, Fig. 19.

The only curve in the figure which is questionable is that which is computed for fixed joints with columns held vertical. In case the columns did incline a little during the test, the result would be a moving upward of the fixed beam curve. This is shown by the dotted diagram which was computed from the following imaginary frame; a 4-ft. beam rigidly fastened between 2-ft. columns, the latter having the same moment of inertia as the beam, and being pinned at the lower ends. The column inclinations computed from the above assumptions are probably in excess of the actual inclinations existing in the test. If this be true, then it follows that the correct moment diagram for a perfectly rigid joint with the column inclinations as they actually were in the test would be somewhere between the two curves.

Up to an applied load of 30,000 lb. the load-strain curves (not shown here) were quite regular. At 88,000 lb. scaling became quite general in the portions of the beam between load points and welded joints. The strain lines extended horizontally, as can be seen in Fig. 17, and stopped abruptly at the stiffeners. As additional load was applied, the deforma-

tion of the beam became more apparent. At 115,300 lb. a few of the lugs were knocked off to afford a view of the top of the fillet welds. As shown in Fig. 18, a small crack had formed at the right-hand joint of the beam. A smaller crack, not photographed, had formed at the other joint. The load was continued to 116,600 lb. with very little increase in the size of the cracks but with much additional crippling of the beam. At this point, which was the maximum load for the beam, an attempt was made to break off the welds at the right joint by vigorously hammering a cold chisel which had been wedged into the crack. This attempt to break the joint failed.

The results of the short beam test are at variance with the results obtained for the longer beams in that the web-welded connections showed unusual strength and tenacity. In one particular the short beam test gave results in harmony with the regular investigation. The percentage of rigidity of the connections was approximately 60 per cent, which agrees rather well with the percentages for the corresponding frames, A-4 and B-4. In spite of the favorable results found in the short beam test the web-welding of short beams is viewed with some apprehension pending further investigation.

Preheating Practices

A. G. WIKOFF[1]

THE fact that all metals expand when heated and contract upon cooling is well understood. When the oxy-acetylene flame is applied to metal, thus raising the temperature greatly at the heated point, the heat spreads through the adjacent metal, which consequently is expanded. When welding is completed, the heat is radiated from the section around the weld and this cooling section contracts. Ductile metals, such as steel, seldom suffer serious internal strain during this expansion and contraction, but non-ductile metals and metals which are comparatively brittle may develop strains which may cause distortion or possibly fracture.

All welders know that the net result of this heating and cooling is determined largely by the shape, size and design of the part being welded. In welding together the pieces of a broken straight bar, for example, the effect of heating and cooling will not cause any difficulty because the bar is entirely free to expand and contract. Very few of the parts that require the welder's attention are as simple as this, however. The welder is constantly called upon to repair broken parts of irregular shape and intricate design or to make welds to join sections of quite different thickness. All such work and, in fact, practically every job that a welder undertakes requires a thorough, practical knowledge of the effects of expansion and contraction.

Provision for the relief of contraction stresses is particularly important where welds are made in parts that are confined at each end, as

[1] Oxweld Acetylene Co.

the spokes of a wheel or pulley. Unless proper precautions are taken, the contraction stresses set up by the cooling weld may be great enough to fracture the spoke.

Intelligent application of preheating affords the most satisfactory method for relieving or avoiding expansion and contraction stresses.

By proper heating, a body expands in all directions, and the heat of welding becomes a minor force. When allowed to cool slowly either in a furnace or in an annealing bin, the contraction will also be in all directions and there will be no resulting stress.

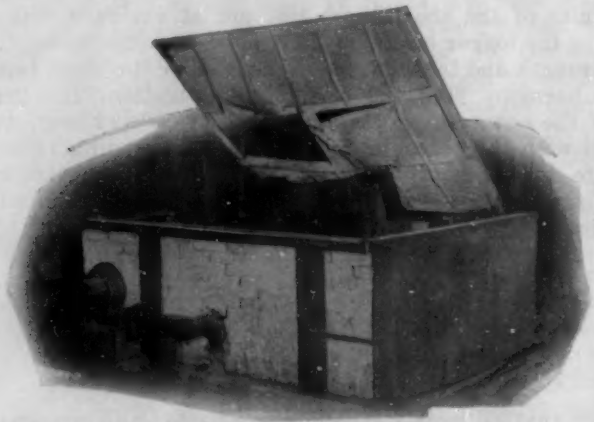


FIG. 1—PERMANENT PREHEATING FURNACE DESIGNED FOR STEADY WORK

FIG. 2—GAS-FIRED TEMPORARY PREHEATING FURNACE

Preheating may also be necessary or advisable for other reasons. The time required for welding a hot piece of metal is considerably less than that required if the metal is cold and has to be brought up to the welding temperature of the oxy-acetylene flame alone. A considerable saving

in oxygen, acetylene and time can therefore be effected by the preheating of the metal with a cheaper fuel. With certain metals, notably cast iron, a weld made on metal properly preheated and slowly cooled can easily be machined, whereas a weld which has been made on cold metal and cooled suddenly afterward will be hard and brittle and very difficult to machine. Proper preheating and subsequent annealing have the additional advantage of relieving strains which may have been in the casting ever since it left the foundry.

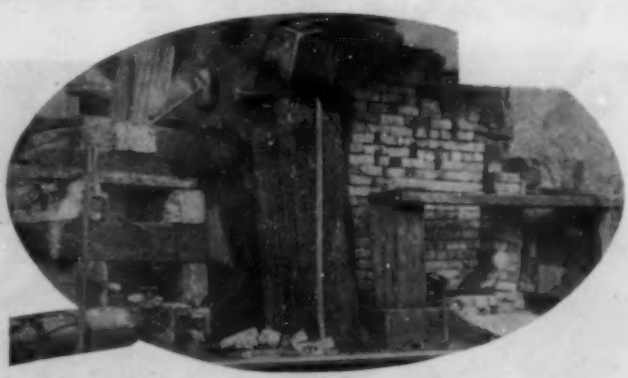


FIG. 3—TEMPORARY CHARCOAL-FIRED FURNACE BUILT AROUND A LARGE CASTING

FIG. 4—CASTING BEING TURNED IN THE PREHEATING FIRE

Various types of apparatus and agencies are employed for preheating, depending, of course, upon whether the preheating is to be general or purely local. The simplest form of local preheating is that which is accomplished by the oxy-acetylene blowpipe flame itself. If the part to be welded is small and the ends free to expand and contract, it is only necessary for the welder to play the blowpipe flame over the metal around the part to be welded before the actual welding is done.

If a forge fire is available, it provides a very convenient means of pre-

heating metal. Care must be taken, however, that the blower is used only enough to keep the fire going and that too much heat is not applied to the metal. Use of an asbestos paper covering is advisable to protect the metal from any possible drafts.

Many types of preheating torches are available which burn inexpensive fuels such as kerosene, fuel oil, natural gas, or city gas. These may be used to accomplish local preheating on large pieces of metal, and, if used in a furnace so that they do not play directly upon the metal, they may be used for general preheating.

FIG. 5—CIRCULAR FURNACE BUILT AROUND BROKEN CAST IRON POT FOR MOLTEN LEAD

For general preheating where many similar large castings are to be welded special permanent preheating furnaces are employed. Preheating furnaces of this type are usually fired with either natural or city gas, as the gas flame is so easily controlled. Fig. 1 shows a permanent preheating furnace used for preheating a regular run of cast iron, steel, and aluminum castings. This type of furnace is also used in the preheating of pipe for bending. The heat may be directed right onto the metal or the preheating may be accomplished indirectly.

The most common preheating agency for general preheating is the temporary fire brick furnace, fired with charcoal. Charcoal is a most effective preheating fuel as it requires no forced draft to burn and its combustion will give a steady, constant temperature. No soot or smoke is given out, which might deposit on the metal and hinder the welding operation. Temporary fire brick furnaces are sometimes fired with gas, oil, or kerosene preheating torches instead of the charcoal. Coke should not be used as a fuel, as it gives too high a temperature and too localized a heat.

The temporary brick furnace is constructed by laying the bricks up around the casting, allowing enough space between the brick wall and

the casting for the fuel and to permit handling the casting if turning is necessary during welding. The lower tiers of the bricks are set with air spaces between them so that the right amount of air may get into the furnace to insure the proper combustion of the fuel. Fig. 2 shows a temporary brick preheating furnace built around a rotary pump casting in a chemical plant. The design of a temporary furnace will vary with each piece to be preheated, depending upon the size, thickness and type of metal. For instance, with very large and heavy cast-

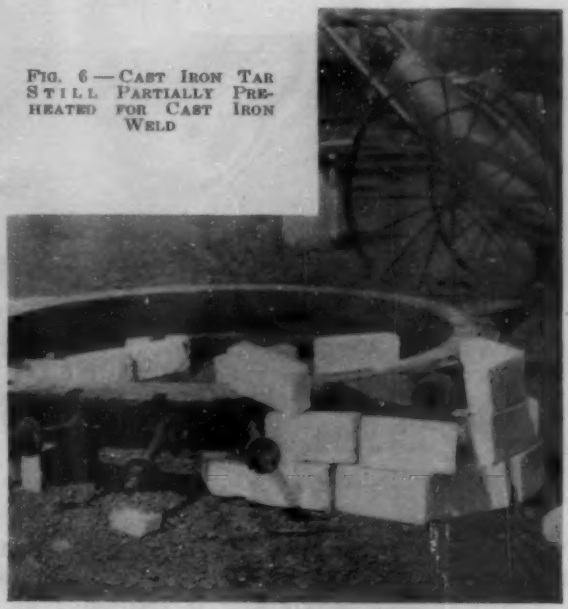


FIG. 6 — CAST IRON TAR STILL PARTIALLY PREHEATED FOR CAST IRON WELD

ings it may occasionally be necessary to use more than one thickness of brick in making the wall of the furnace. The most usual covering for a temporary brick preheating furnace is a sheet of asbestos paper supported on bars or light angle iron placed across the top of the furnace. Several small holes are punched in the paper to allow for the escape of the products of combustion. The section of the metal to be welded should be near the top of the preheating furnace, so that when the proper temperature has been reached in the preheating the asbestos paper covering may be drawn back sufficiently to permit the welding to be carried on while the metal remains within the furnace. After the weld is finished, this covering may be put back over the whole furnace to protect the metal from sudden cooling.

Cast Iron

The most commonly used metal which requires preheating before welding is cast iron. The physical properties and the methods of treating this metal should therefore be considered first in any discussion of the subject of preheating.

In ordinary gray iron castings, free carbon is present in the form of graphite flakes. This graphitic carbon gives gray cast iron its characteristic properties such as machineability and is also partly responsible for its susceptibility to fracture under sudden shock. The whole theory of successful cast iron welding is based upon retaining the gray iron structure in the finished weld. Cast iron welding rod of high silicon and adequate carbon content is of paramount importance in securing this. Proper preheating and annealing are also vital. When gray iron is melted the graphite flakes disolve. If the molten iron is cooled quickly

FIG. 7—SIDE FRAME OF SHEAR PREPARED FOR PREHEATING

the carbon does not separate out again as graphite flakes and the result is hard, unmachineable white cast iron. The slow cooling or annealing necessary for the formation of graphite is facilitated by proper preheating prior to welding. Preheating of gray iron castings is also essential in order to avoid expansion and contraction strains as noted previously.

In using cast iron welding rod for cast iron welding, the whole casting must be preheated in a furnace, or a sufficiently large section of the casting must be preheated locally. Fig. 3 shows a large temporary charcoal fired preheating furnace built around a casting and Fig. 4 shows the casting being turned in the fire. In Fig. 5 a large cast iron pot for melting lead is resting in a temporary furnace which has been built around it. The preheat was supplied by a kerosene preheating torch. Fig. 6 shows a cast iron tar still of which only the broken section is being preheated for welding. The rest of the casting is protected from the heat by the small brick enclosure. A large cast iron side frame of a shear is shown in Fig. 7 with the broken end enclosed in a temporary furnace. A charcoal fire brought this section up to the proper temperature so that it could be welded with cast iron rod through the asbestos paper covering, Fig. 8. The completed cast iron weld may be seen in Fig. 9. Local preheating of a small part by the blowpipe flame

is shown in Fig. 10. A broken lug is being welded to a cast iron pump base. In Fig. 11 two welders are shown reclaiming a disc crusher casting in a temporary furnace. The preheat was furnished by kerosene torches.

A kerosene preheating torch furnished the preheat for the cast iron

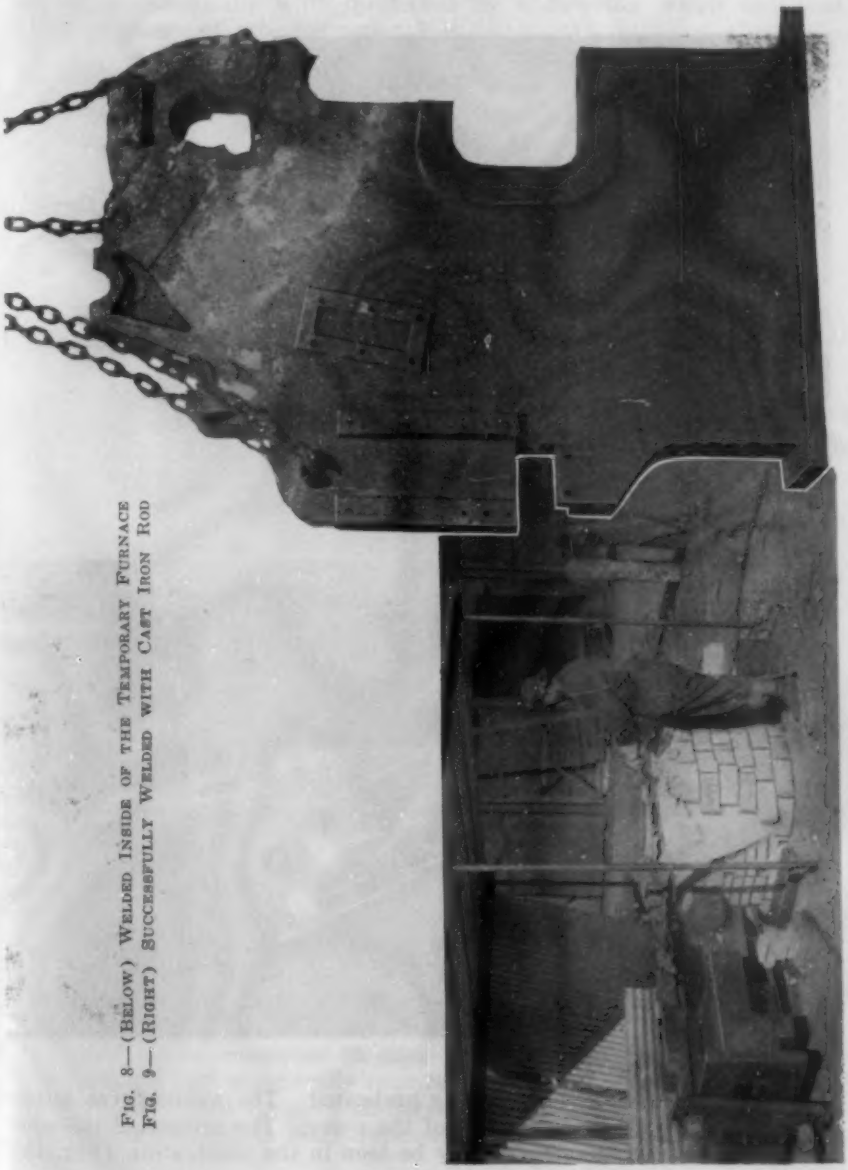


FIG. 8—(BELOW) WELDED INSIDE OF THE TEMPORARY FURNACE
FIG. 9—(RIGHT) SUCCESSFULLY WELDED WITH CAST IRON ROD

repair shown in Fig. 12. A large punch press was surrounded by the brick furnace and the preheating flame played through an opening.

In Fig. 13 a large rack for holding a roll of asbestos paper may be seen. This has been found very convenient by a contract welding shop where numerous cast iron repairs are made which necessitate preheating. Fig. 14 shows the asbestos paper being used to cover up the brick

FIG. 10 — (LEFT) SMALL PARTS LOCALLY PREHEATED WITH BLOWPIPE

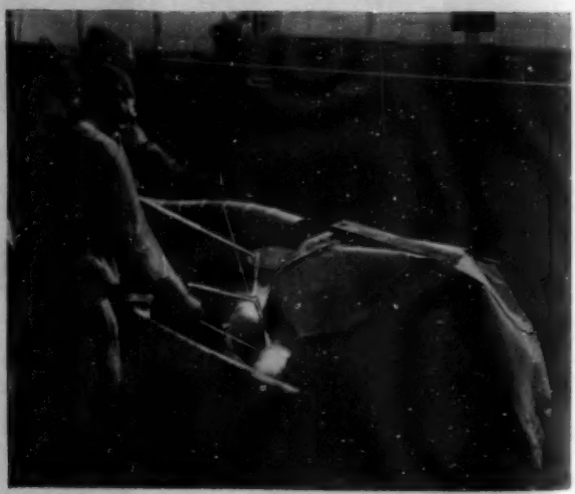


FIG. 11 — DISC CRUSHER IN MAKE-SHIFT FURNACE

furnace and a casting that is being preheated. The welding was accomplished through a hole in the top of the cover. The completed job when the covering is being removed may be seen in the illustration (Fig. 15).

Fig. 16 shows a typical layout for preheating iron castings of various sizes and shapes. A large section of the shop has a fire brick floor, and piles of loose fire brick are at hand, so that a furnace of any size may be quickly built. Another contract shop designed and uses the gas fired permanent furnace shown in Fig. 17 for various kinds of iron castings.

The reclamation of a large cast iron filter for lubricating oil was recently accomplished in a Western refinery. On this job, the casting was preheated locally while protected by a covering which consisted chiefly of sheets of asbestos paper, Fig. 18.

Fig. 12—Preheating a Large Punch Press

The welding of cast iron with cast iron welding rod, however, is gradually being superseded in many cases by the use of bronze-welding. The melting point of bronze is lower than that of cast iron so it is not necessary to bring the cast iron up to such a high temperature for the welding, thereby lessening the temperature necessary for preheat and also resulting in economies in oxygen and acetylene. The use of local preheat is all that is generally required for the successful bronze-welding of cast iron. This means that large pieces of machinery need not be dismantled in order to make repairs. Fig. 19 shows the local preheat-

ing of a huge press in an automobile factory, and Fig. 20 shows the actual bronze-welding. It will be noted that the rest of the machinery is protected from the heat by asbestos paper. The press was reclaimed in place, no dismantling being necessary. Through the use of bronze-welding it is possible to weld crank cases and cylinder blocks without removing the engine from the automobile. Fig. 21 shows the bronze welding of a truck cylinder block, which was locally preheated and protected from drafts with asbestos paper. A dull red heat is all that is required of cast iron before bronze-welding. A casting hot enough to char a piece of newspaper is ready to weld with bronze.

FIG. 13—RACK FOR ASBESTOS PAPER USED BY SHOP IN PREHEATING PRACTICE

FIG. 14 (LEFT)—COVERED BY ASBESTOS PAPER AFTER WELDING

Cast Aluminum

Cast aluminum is a metal which always requires a certain amount of preheating before any welding is attempted. In most cases, general preheating is employed and the whole casting is placed in a preheating furnace and brought up to the correct heat. More care is necessary with the preheating of cast aluminum than almost any other metal. Aluminum is relatively weak when heated and overheating may cause the casting to collapse of its own weight. For this reason, too, attention should be given to providing adequate support for the casting in the furnace. The

temporary fire brick preheating furnace, previously described, is the most satisfactory method for the general preheating of cast aluminum. The control of the preheat and the proper regulation of the draft require the constant attention of the operator owing to the peculiar properties of this metal. If this type of welding is a routine matter in production work, a permanent preheating furnace is recommended. Preheating should be done very slowly in all cases, and the casting temperature must

FIG. 15—UNCOVERING THE COMPLETED JOB

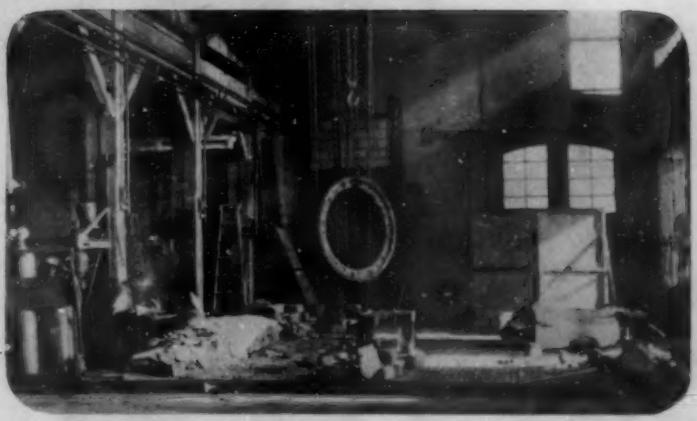


FIG. 16—AN IDEAL LAYOUT FOR BUILDING TEMPORARY FURNACES

be carefully watched. Cast aluminum does not show red heat before reaching the molten state. It becomes pasty, and it is difficult to tell merely from the surface appearance that the casting has reached the proper temperature. When cast aluminum has become hot enough to char sawdust, it is hot enough to weld. On small pieces, local preheating with a blowpipe will be sufficient to bring the edges to the proper heat. The blowpipe flame should never be held less than 1½ or 2 in. from the metal and should be constantly moved over the part, so as to insure uniform heating.

The repair of a cast aluminum automobile crank case is one of the most frequent cast aluminum welds requiring preheating. General preheating in a temporary furnace or by a preheating flame, which con-

tinues to play on the casting during welding, is nearly always employed. Any badly broken aluminum casting of this type should be placed in the furnace with the middle of the break horizontal and near the top of the furnace, so that the welding may be carried on through the asbestos covering opening. A cold casting should be brought up to the proper temperature very slowly, the average time being about 30 min. Besides

FIG. 17—INTERESTING TYPE OF PERMANENT FURNACE IN SHOP

FIG. 18—OIL FILTER PREHEATED UNDER ASBESTOS PAPER

the test with sawdust, the condition of the casting may be tested by using the steel rod to scrape the surface of the metal to make sure that it is not becoming too soft. In the case shown in Fig. 22 only a small break occurred so that complete preheating was unnecessary.

Monel Metal and Pure Nickel Castings

Castings of pure nickel or Monel metal, which is a natural alloy of nickel and copper, are quite sensitive to sudden temperature changes. General preheating in a furnace is essential to the successful welding of these castings. As in the case of cast aluminum, it is necessary to give the casting proper support while it is in the preheating furnace, so that it will not distort or collapse under its own weight. Pure nickel or Monel castings will, however, show a dull red heat when a tempera-

ture of about 1200 deg. F. is reached, which is the proper temperature to which the casting should be brought before welding is started, in order to eliminate both uneven heating strains and excessive oxidation which may result from the local overheating of the metal. A dull red heat should be maintained as steady as possible during the welding operation. The lack of strength of the metal when hot should also be kept in mind.

FIG. 19 — LOCAL PREHEAT PERMITTED BRONZE WELDING IN PLACE

FIG. 20—APPLYING THE BRONZE

Bronze Castings

Bronze castings vary so widely in composition and metallurgical characteristics that it is difficult to prescribe a regular procedure to apply to all of them. In general, whenever the casting is of considerable size, preheating should be used. This preheating is general and is usually accomplished in a furnace. Fig. 23 shows a fractured cast bronze

gear of large size which is being welded with high strength bronze rod after having been preheated in a furnace. It will be noted that the preheat is being retained by an asbestos paper covering. It was necessary to preheat this gear for about three-quarters of an hour before welding began. Then the operator was able to complete the weld in a little over an hour. Bronze castings become a dull red color when sufficient preheat has been applied to allow the welding to begin. Fig. 24 shows a famous bronze bell cast originally by the Paul Revere Foundry in Boston and recently restored by the use of manganese bronze welding rod. The preheating of this bell for the welding was accomplished by digging a hole in the dirt floor of a welding shop and building a charcoal fire. The top of the hole was covered with asbestos paper to prevent any

FIG. 21—BRONZE-WELD IN CAST IRON CYLINDER BLOCK OF TRUCK

draft from striking the casting. A job of this sort, however, should never be attempted by anyone but an expert, as an intimate knowledge of the proper procedure for welding bronze bells is essential if the bell is to ring true. In this repair and the repair of the gear just mentioned, proper preheating was essential in order to maintain both of these castings in correct alignment.

Cast Steel

Steel is a metal which rarely requires any preheating, because of its ductility and toughness. A good steel casting will bend considerably before breaking, and an examination of any broken steel casting will show that it has undergone much distortion before it finally broke. Small steel castings require no preheating at all. Larger castings, however, with thick walls of such design that localized heat of welding might set up strains in the steel, should be preheated to a bright red color before the welding is started. The design of the casting will determine largely whether the preheating should be general or local. The principles for preheating cast steel are the same as outlined above for the preheat-

ing of cast iron. Fig. 25 shows the cylinder of a compressed air paving breaker with a broken leg, after welding. Owing to the peculiar shape of this steel casting the part to be welded was preheated in a small furnace so that as the broken member was welded, the expansion took place equally throughout the casting and the contraction which followed was equalized and caused no strain.

FIG. 22 — GAS TORCH MAINTAINS PREHEAT DURING CAST ALUMINUM WELDING

FIG. 23 — WELDING PREHEATED BRONZE GEAR

Alloy Steels

Alloy steels contain, in addition to carbon, one or more metals such as nickel, chromium, vanadium, manganese or molybdenum. For many purposes these alloy steels are superior to the ordinary carbon steel. As the full strength of these steels is developed only after they have been heat-treated under special conditions, the welding of these alloy steels should be attempted only by shops having adequate facilities for correct heat-treatment.

Before Cutting

Preheating should precede the cutting of high carbon and alloy steels with oxy-acetylene blowpipe. Steels with a carbon contact of 0.25 per cent or higher should be preheated before any oxy-acetylene cutting is attempted, in order to prevent any harmful expansion and contraction stresses on the metal near the cut edge. High carbon steel for example should be preheated to a temperature of about 1100 deg. F., so that,

FIG. 24 — WELDED WITH MANGANESE BRONZE IN PREHEATING FURNACE

FIG. 25 — CAST STEEL CYLINDER OF COMPRESSED AIR PAVING BREAKER

while it is being removed from the preheating furnace to be cut, the temperature will fall to approximately 1000 deg., which is the proper temperature for cutting.

The chief reason for preheating before cutting is that a physical change occurs on the cut edge of a piece of a steel which is cut cold. This change is unnoticeable on low carbon steels, which therefore require no preheating. The change consists in a hardening effect along the face

of the cut extending from 1/16 to ⅛ in. into the metal and is caused by the rapid transfer of heat from the portion being cut to the cold adjacent metal. In the high carbon and alloy steels, this zone indicates a gradual change in structure. The sudden cooling of the cut edge sets up contraction stresses as the face of the cut has been heated during the cutting by the blowpipe to a sufficiently high temperature to change the grain structure. High carbon and alloy steels do not possess sufficient ductility after being suddenly heated and cooled to take care of these expansion and contraction stresses, so that doing the cutting on a hot slab is the simplest and best way of handling the problem efficiently and economically.

The removal of risers in a steel foundry is a good example of a heavy oxy-acetylene cut in which preheating has been found economical. The heat remaining in the castings as they are removed from the molds conveniently furnishes all the preheat that is required. This method of riser removal is specially recommended in the case of high carbon and alloy steel castings.

Summary

The methods of and reasons for the preheating of certain metals before welding or cutting have been outlined above. It may be noted that the reasons for this practice are both metallurgical and also economical in some instances. Any study of the subject of preheating will serve to emphasize the importance to welders of a knowledge of the behavior of metals when heated. The difficulties which were encountered in the past in welding some metals were solved largely by an understanding of heat control. The equalization of expansion and contraction stresses and the physical properties of the various metals are matters which the welding expert understands and which act as guides in preheating practice.

The correct procedures for the preheating of the metals most commonly encountered in general welding practice have been discussed in this article, although there are many other metals which require preheating before they are welded.

If the practices outlined above are followed, little difficulty caused by expansion or contraction stresses will be experienced in welding these metals.

The Theory of Stresses in Welds*

PART I

By Leon C. Bibber†

I—Introduction

MUCH has been written in the publications of this Society about the *practice* of welding, but very little about the *theory*. The strength of welding as affected by *quality* is without the province of this discussion; it is the purpose of this paper to advance certain theories regarding the strength of welds as affected by the *quantity* and *disposition* of the weld metal, and later, on the basis of these theories, to evolve design data and tables that will enable designers and draftsmen to use this information even though they may not be familiar with the evolution of the theory.

The future of welding is in the hands of the designers, and all design is based on theory. It is hoped that the ideas advanced herein will stimulate thought and discussion that will lead ultimately to the establishment of a welding theory that will take its place along with the other accepted theories of the strength of materials. Once such a theory has proved its worth it can be accepted by insurance companies, classification societies, and governmental authorities, and no design of any importance can become a reality without the approval of these bodies that safeguard the life and property of the public.

The theory will remain unchanged by the progress of welding, it will hold for mild steel, high-tensile steel and the non-ferrous metals; all that will be needed will be to apply the proper values. Furthermore, it will hold for the different kinds of stress, for instance, the theory of tension will be the same for static tension, for tension impact, for tension fatigue, and for tension corrosion fatigue, the only thing that will change will be the working values.

II—General Considerations

The uniformity of rolled steel is assured by chemical and physical testing, and by long experience in rolling-mill practice, and the values of ultimate strength, elastic limit, etc., as determined by test may be used with confidence as a basis of design. Since all efficiencies are relative it is necessary to have some standard of comparison. A mild steel of the physical characteristics listed in the following table has been selected as the basis of all efficiencies, and will be called the *standard base metal*.

A weld deposited with a bare, mild-steel electrode or welding-rod on mild steel, and having the physical characteristics listed in the table will

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†Senior Welding Engineer, Bureau of Construction and Repair, Navy Department.

be called the *standard weld.* These characteristics are representative of a good weld made under good conditions, and will be called the *standard values.*

Unfortunately, however, the values of one weld are no indication of those of another, and by reason of this lack of uniformity a lower value than is indicated by tests must be used to represent the work of the average welder under average conditions. These arbitrary values will be called the *design values.*

TABLE I

Assumed Physical Properties

1. Weld Metal, *Standard Values.*		
(a) Ultimate Strength in Tension	57,000	lbs./in.²
(b) Yield Point in Tension	43,000	lbs./in.²
(c) Modulus of Elasticity in Tension	29,000,000	lbs./in.²
(d) Ultimate Strength in Compression	63,000	lbs./in.²
(e) Yield Point in Compression	29,000	lbs./in.²
(f) Ultimate Strength in Transverse Shear	41,000	lbs./in.²
(g) Yield Point in Transverse Shear	22,500	lbs./in.²
(h) Modulus of Elasticity in Transverse Shear	11,000,000	lbs./in.²
2. Bond Between Weld Metal and Base Metal, *Standard Values.*		
(a) Ultimate Strength in Tension	40,000	lbs./in.²
(b) Ultimate Strength in Transverse Shear	41,000	lbs./in.²
3. Weld Metal, *Design Values.*		
(a) Ultimate Strength in Tension	36,000	lbs./in.²
(b) Ultimate Strength in Compression	36,000	lbs./in.²
(c) Ultimate Strength in Transverse Shear	26,000	lbs./in.²
4. Bond Between Weld Metal and Base Metal, *Design Values.*		
(a) Ultimate Strength in Tension	25,000	lbs./in.²
(b) Ultimate Strength in Transverse Shear	25,000	lbs./in.²
5. Base Metal, *Standard Values.*		
(a) Ultimate Strength in Tension	60,000	lbs./in.²
(b) Yield Point in Tension	30,000	lbs./in.²
(c) Modulus of Elasticity in Tension	29,000,000	lbs./in.²
(d) Ultimate Strength in Compression	60,000	lbs./in.²
(e) Yield Point in Compression	30,000	lbs./in.²
(f) Ultimate Strength in Transverse Shear	50,000	lbs./in.²
(g) Yield Point in Transverse Shear	25,000	lbs./in.²
(h) Modulus of Elasticity in Transverse Shear	11,000,000	lbs./in.²

There are three elements in the strength of any weld, first, the bond between the weld metal and one member, second, the weld metal itself, and third, the bond between the weld metal and the other member. Deficiency in any one of these three elements may cause failure of the joint, and any discussion of the strength of welds must consider all three. Since the bond is liable to suffer as much or more from improper technique than is the weld metal itself, it is necessary that there be both standard and design values for the bond.

As shown in the table, 57,000 lb. per sq. in. has been selected as the standard value for the ultimate strength of weld metal in tension. 36,000 lb. per sq. in. is widely accepted as the design value for the ultimate strength of weld metal in tension, and as this design value is about 63 per cent of the standard value, all other design values are derived by reducing the standard values in about that proportion.

It is appreciated that the above arbitrary selections are only a matter of opinion, and that as such they are subject to revision, but it was necessary to select some values to develop the theory, and if these are not considered satisfactory, others can be chosen that will represent the consensus of opinion of the Society. They were decided upon, however, after consideration of all available data.

A weld of exact geometrical shape, without penetration, will be called the *theoretical weld.*

A weld that approximates the exact geometrical shape, but has penetration will be called the *actual weld.*

The depth of penetration will vary with a number of factors but a constant penetration of one sixteenth of an inch will be assumed for all welds. No weld should have less than this, and more will be advantageous.

In the mathematical discussions to follow, all sections will be assumed to be 1 in. in width, and therefore width does not enter into the equations.

The leg which is parallel to the force in the members will be referred to as the *parallel leg,* and the leg which is normal to the forces will be referred to as the *transverse leg.*

In weld metal, as in any other metal, there are two kinds of failure, *simultaneous failure* and *progressive failure.*

Simultaneous failure is rupture that occurs at the same time over the entire critical section.

Progressive failure is rupture that occurs in one part of the critical section, and then because of the cumulative reduction of area, progresses rapidly throughout the rest of the section until total failure results.

Any of the three kinds of stress may cause progressive failure, and the stresses causing such failure will be referred to as *progressive tension, progressive compression,* or *progressive shear.* When these stresses are unqualified, they will refer to simultaneous stresses.

III—Fillet Welds In Tension

A. Theoretical Welds.

(1) 45-Degree Fillets.

Fig. 1 shows a theoretical full fillet weld under a tension force P. The lines of stress show the passage of the force from member A, through the weld, and into member B. With respect to member A the force is wholly concentric, with respect to B, it is wholly eccentric.

Consider the conditions of equilibrium in the parallel leg SU. In order to develop the force P, member B must pull on the weld with a resultant force R, which equals

$$\frac{P}{\cos 45^\circ} = \frac{P}{.707} = 1.41\ P.$$

This resultant is transmitted through the throat of the weld to the transverse leg SV, where it is resolved back into the force P in member A.

At the leg SU, the force P is a shear force, and the force Q is tension, at the leg SV, the force P is tension and the force Q is shear. Therefore the legs of a fillet weld in tension are subjected to combined shear and tension.

Since the above forces are in equilibrium, the presence of any other force would upset this equilibrium. The resultant R passes through the throat of the weld, and since no other force can be there, the force in the throat of a fillet weld in tension is pure tension. This fact has also been demonstrated by means of experiments with transparent models and polarized light.

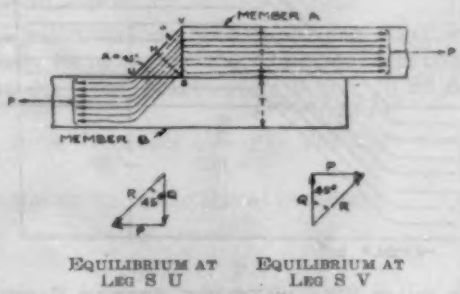

FIG. 1. THEORETICAL FILLET WELD IN TENSION

The tensile stress in the base metal is $\frac{P}{T}$. The area of the throat SH, is T cos 45° = .707 T. The stress in the throat equals

$$\frac{R}{SH} = \frac{1.414\,P}{.707\,T} = \frac{2\,P}{T}$$

Therefore the stress in the throat of a theoretical full fillet weld in tension is exactly twice the stress in the base metal. In other words, if the weld metal were equal in value to the base metal, the efficiency of a full theoretical fillet weld in tension could not exceed 50 per cent, or if there were no weld at all and solid metal were cut to the proper shape, then the efficiency of such shape could not exceed the above value.

The legs SU and SV are subject to both shear and tension. The formula for combined or principal stress is

$$\text{Principal stress} = \frac{\text{tensile stress}}{2} + \sqrt{(\text{shear stress})^2 + \frac{(\text{tensile stress})^2}{4}}$$

The tensile stress is $\frac{P}{T}$, and the shear stress is likewise $\frac{P}{T}$, and therefore the combined stress is $\frac{1}{2}\frac{P}{T} + \sqrt{\left(\frac{P}{T}\right)^2 + \frac{1}{4}\left(\frac{P}{T}\right)^2} = 1.618\,\frac{P}{T}$

This stress is a tensile stress and acts in the direction R, and the area upon which its value is measured must be perpendicular to R. But the planes of the bonds SU and SV are not perpendicular to R, and they are the planes of weakness; any departure from them would carry the stress out into the stronger base or weld metal. Probably what actually happens is this:

The fracture is not a plane, but a jagged irregular area as shown diagrammatically by Fig. 2.

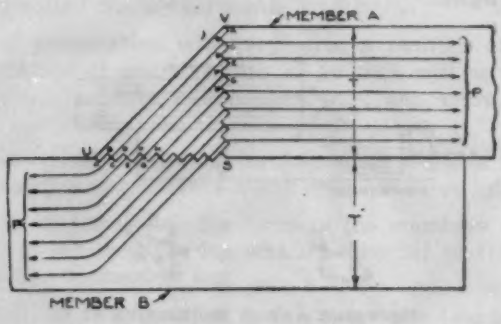

FIG. 2. THEORETICAL FILLET WELD IN TENSION, SHOWING HYPOTHETICAL PLANES OF FAILURE IN LEGS

On facets BC, DE, FG, etc., normal to R, failure occurs by tension, and on facets AB, CD, EF, etc., parallel to R, failure occurs by shear. This is not an attempt to depict a crystalline formation, the facets may be microscopic in size or they may be larger than those shown, and they may be independent of crystalline boundaries. If they form right angles, the area of the bond, that is, the summation of AB, BC, CD, etc., will be a maximum and will be $\frac{T}{\cos 45^\circ} = 1.41\ T.$

The design value of the ultimate strength of the bond in tension is 25,000 lb. per sq. in., and in shear, is 25,000 lb. per sq. in. The areas in shear and tension are equal and therefore the value to be used for the entire bond is the mean between the two, or 25,000 lb. per sq. in.

Since the combined stress is 1.618 times the stress in the base metal, the maximum force P that can be transmitted to the base metal will be the mean value of the bond times the maximum area of the bond, divided by 1.618 or $\frac{25{,}000 \text{ lb. per sq. in.} \times 1.41\ T}{1.618} = 21{,}800$ lb. per sq. in. T. The efficiency of the legs equals $\frac{21{,}800 \text{ lbs./in.}^2\ T}{60{,}000 \text{ lbs./in.}^2\ T} = 36.3$ per cent.

The stress in the throat of the weld is double that in the base metal, and the design value of the ultimate strength of weld metal in tension

is 36,000 lb. per sq. in. and therefore the stress that can be transmitted by the throat to the base metal is 18,000 lb. per sq. in., and the value of the base metal at this stress is 18,000 lb. per sq. in. T. The value of the base metal itself is 60,000 lb. per sq. in. T, and therefore the design efficiency of a theoretical full fillet weld in tension becomes

$$\frac{\text{18,000 lb. per sq. in. T}}{\text{60,000 lb. per sq. in. T}} = 30 \text{ per cent.}$$

From the above discussion it can be seen that the strengths of the elements of the theoretical 45 deg. fillet weld are not balanced, that the legs are capable of being somewhat stronger than the throat.

(2) *The Influence of Shape*

The above discussion was confined to 45 deg. fillet welds. But welds need not necessarily be of that shape. Consider the conditions when the angle α is not 45 deg., but ranges from 0 deg. to 90 deg.

The resultant force R equals $\frac{P}{\cos \alpha}$. The area of the throat SH is T cos α. The tensile stress in the throat becomes

$$\frac{\frac{P}{\cos \alpha}}{T \cos \alpha} = \frac{P}{T} \times \frac{1}{(\cos \alpha)^2} \qquad \text{II}$$

That is, the tensile stress in the throat of a theoretical full fillet weld under tension varies as the inverse square of the cosine of the angle α.

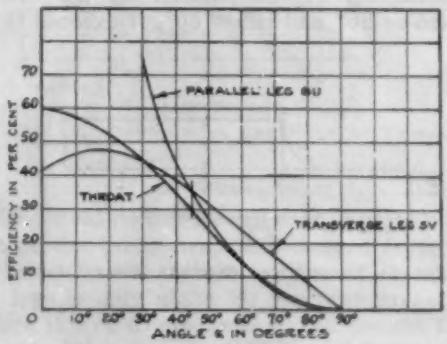

FIG. 3. THE RELATION BETWEEN THE EFFICIENCY OF THE ELEMENTS OF A THEORETICAL FULL FILLET WELD IN TENSION AND THE ANGLE α

In the parallel leg SU, the area of the leg $= \frac{T}{\tan \alpha}$. The shear force is P, and the shear stress $= \frac{P}{\frac{T}{\tan \alpha}} = \frac{P}{T} \tan \alpha$

The tension force = Q = P tan α. The area of the leg = $\frac{T}{\tan \alpha}$. The tensile stress = $\frac{P \tan \alpha}{\frac{T}{\tan \alpha}} = \frac{P}{T} (\tan \alpha)^2$

$$\text{The combined stress} = \frac{P}{T} \tan \alpha \left[\frac{\tan \alpha}{2} + \sqrt{1 + \frac{(\tan \alpha)^2}{4}} \right] \quad \text{III}$$

In the transverse leg, the area = T. The tension force = P. The tensile stress = $\frac{P}{T}$. The shear force = Q = P tan α.

$$\text{The shear stress} = \frac{P \tan \alpha}{T} = \frac{P}{T} \tan \alpha.$$

$$\text{The combined stress} = \frac{P}{T} \left(\tfrac{1}{2} + \sqrt{(\tan \alpha)^2 + \tfrac{1}{4}} \right) \quad \text{IV}$$

Fig. 3 is plotted from equations II, III and IV and shows the relation between the efficiency of the elements and the angle α. When α = 0° the fillet weld becomes a butt weld, and the efficiency of the throat is merely the ratio of the strength of the weld metal to the strength of the base metal or

$$\frac{36{,}000 \text{ lb. per sq. in.}}{60{,}000 \text{ lb. per sq. in.}} = 60\ \%.$$

The maximum efficiency of the parallel leg SU drops rapidly from infinity at 0° to zero at 90° and below 60°, it exceeds that of the throat.

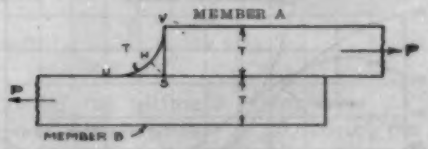

Fig. 4—Theoretical Concave Fillet Weld

At 0° the maximum efficiency of the transverse leg is the ratio of the strength of the bond to that of the base metal or

$\frac{25{,}000 \text{ lb. per sq. in.}}{60{,}000 \text{ lb. per sq. in.}} = 41.7\%$. The efficiency of this leg exceeds the throat at angles above 30°, and equals that of the parallel leg at 45°. It will be noted that at 45° the two legs are slightly stronger than the throat.

If the face of the weld be made concave as shown in Fig. 4, the area of the throat will become (1.41 T) — T = .41 T. R remains 1.41 P as before, and the stress in the throat becomes

$\frac{1.41\ P}{.41\ T}$ or $3.44\ \frac{P}{T}$, that is, 3.44 times the stress in the base metal.

This would permit a value of

$\frac{36{,}000\text{ lb. sq. in. T}}{3.44} = 10{,}450$ lb. sq. in. T in the base metal, and the theoretical efficiency of such a fillet is $\frac{10{,}450\ T}{60{,}000\ T} = 17.4\%$.

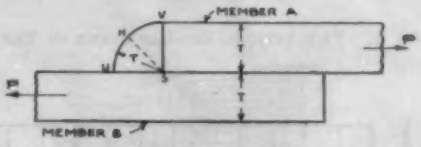

FIG. 5. THEORETICAL FILLET WELD REINFORCED BY INCREASING THROAT

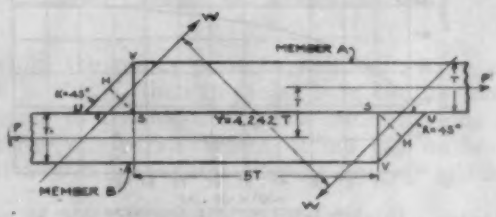

FIG. 6. THEORETICAL LAP JOINT IN TENSION

If the face of the weld be made convex as is customary with reinforced welds, the depth of the throat will be increased from .707 T to T, or 41 per cent. Referring to the curves in Fig. 3, it can be seen that the strength of the legs is only about 20 per cent greater than that of the throat. Therefore it would appear that full benefit from reinforcement cannot be obtained with theoretical fillet welds.

(3) *The Fillet Welded Lap Joint.*

In the preceding discussion the effect of eccentricity was not considered. Let us now investigate welds that will be affected by this condition.

The fillet-welded lap joint shown in Fig. 6, is not in equilibrium. The forces P balance each other, but the couple PT is not balanced. Under load the joint would move and the forces P would try to line them-

selves up and pass through the center of the joint as shown by Fig. 7, thus causing the couple PT to vanish.

The welds would have to exert pull enough not only to balance the forces P but also to bend the members A and B and hold them in the bent position. By the time this had occurred the joint would have taken

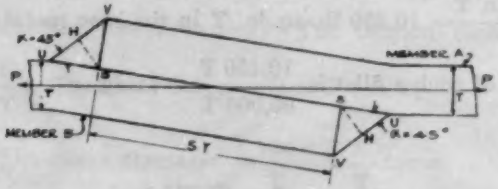

FIG. 7. THEORETICAL SET-LAP JOINT IN TENSION

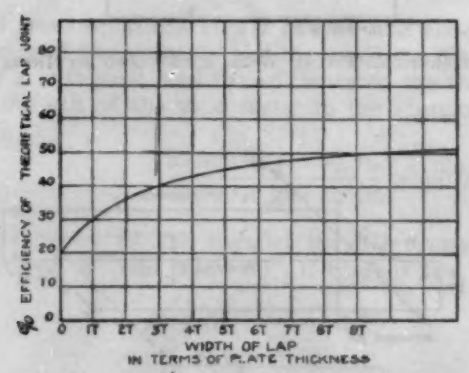

FIG. 8. THE RELATION BETWEEN WIDTH OF LAP AND THE THEORETICAL EFFICIENCY OF A LAP JOINT IN TENSION

a permanent set, that is, the bending stresses in the base metal would have exceeded the elastic limit of the material. It would not be advisable to permit this distortion in any structure, and therefore the joint will be considered as shown in Fig. 6.

The only way that forces can act in the joint is through the welds, and therefore the couple PT will be transferred to the throat of the welds. From the geometry of the figure the lever arm Y of the new couple will be found to be 4.242 T.

$$W \times 4.242\,T = PT \qquad W = \frac{PT}{4.242\,T} = .236\,P$$

The only thing that can balance a couple is another, equal and opposite to the first, and therefore, forces equal to 23.6 per cent of the force P

must be added to those already in the welds. On the assumption that the load is equally divided between the two welds, the resultant in each due to the force P is $\frac{.5\,P}{.707} = .707\ P$. Hence the additional load on the welds due to eccentricity is $\frac{.236\,P}{.707\,P} = 33.3\%$.

Fig. 8. shows how the efficiency of theoretical lap joints varies with width of lap. As the width of lap decreases, the efficiency drops off rapidly. This we know to be true from experience. A width of lap of about five or six plate thickness is a good practical lap; above these widths the curve is flattening out and the gain in efficiency may not be worth the added material necessary to secure it.

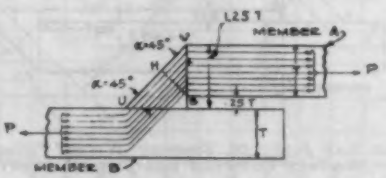

FIG. 9. THEORETICAL FILLET WELD IN TENSION WHEN THERE IS NOT METAL-TO-METAL CONTACT BETWEEN THE FAYING SURFACES

If a set be put in the plates prior to welding, giving a set-lap joint such as is shown in Fig. 7, then the joint is in equilibrium; there is no couple to add forces to the welds and the latter may be considered as developing their full efficiency. Width of lap has no bearing on such a joint; theoretically it is equally efficient for all widths.

(4) *The Influence of Not Obtaining Metal-to-Metal Contact*

The faying surfaces of lapped and strapped joints are not always in absolute metal-to-metal contact. This is particularly true of welded joints where bolts may not be used at all, or bolt holes are not as abundant as is the case with riveted joints. An exaggerated example of this is shown in Fig. 9.

It can be seen that the weld size has increased from T to 1.25 T, but the length of the transverse leg SV has not been affected, neither has the throat SH. The stresses must pass through the throat of the weld in the same manner as indicated in Fig. 1, and therefore, not obtaining metal-to-metal contact between the faying surfaces of such a weld, does not influence the strength of the weld.

When the metal-to-metal contact is not obtained between the faying surfaces of a lap joint, however, the strength of the joint is affected. Fig. 10 shows such a joint.

It will be seen that the unbalanced couple has been increased from

PT to 1.25 PT and that the distance between the forces W has been increased to 4.419 T. The additional force W becomes

$$W \times 4.419\ T = 1.25\ PT$$

$$W = \frac{1.25\ PT}{4.419\ T} = .283\ P$$

The efficiency of the joint therefore reduces from 45 per cent to 42.85 per cent. Hence for a relatively large opening between the plates, there is but a small drop in theoretical efficiency, so that the ordinary small

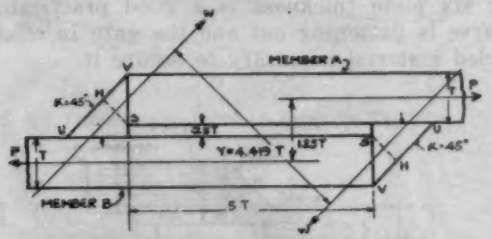

FIG. 10. THEORETICAL LAP JOINT IN TENSION WHEN THERE IS NOT METAL-TO-METAL CONTACT BETWEEN THE FAYING SURFACES

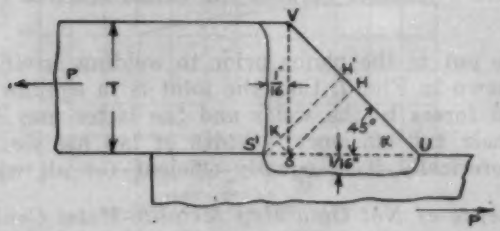

FIG. 11. ACTUAL FILLET WELD IN TENSION

space between the faying surfaces of welded joints is no great cause for alarm.

B. ACTUAL WELDS IN TENSION

(1) 45-*Degree Fillets*

Thus far we have considered theoretical welds only, and neglected the effect of penetration. Let us now consider the actual welds, as shown by Fig. 11.

It will be seen that the theoretical throat SH has been shifted toward the transverse leg, and been lengthened into the actual throat S'H'.

$S'H' = SH + KS'$. $SH = T \cos \alpha$. $KS' = SS' \sin \alpha$.

Therefore $S'H' = T \cos \alpha + SS' \sin \alpha$, and when $\alpha = 45°$ $S'H' = (T + 1/16 \text{ in.}) \cos \alpha = T \cos \alpha + .0442$ in.

Therefore the throat of all fillet welds has been increased by the value 1/16 in. sin α, and the throat of all 45 deg. fillets by the constant .0442 in. Since the penetration is assumed to be a constant value, the benefit that it confers will be relatively greater for the smaller welds. In Fig. 12 the increase in strength of actual welds over the theoretical is plotted against the ordinary range of sizes.

In the lower range the curve rises rapidly to infinity, as shown by the dotted line; this would indicate that tremendous increases in strength

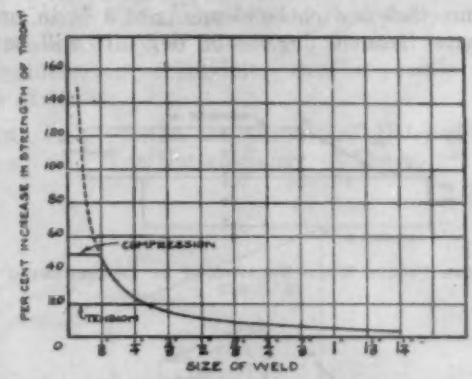

FIG. 12. THE RELATION BETWEEN THE INCREASE IN STRENGTH OF THE THROAT OF THE ACTUAL WELD OVER THE THEORETICAL, AND THE SIZE OF WELD

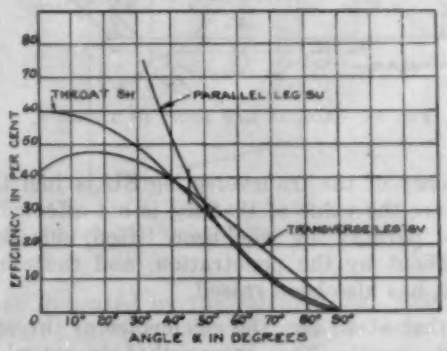

FIG. 13. THE RELATION BETWEEN THE EFFICIENCY OF THE ELEMENTS OF AN ACTUAL ⅝ IN. FILLET WELD IN TENSION, AND THE ANGLE α

might be expected from the smaller welds. Referring again to Fig. 11 it will be seen that the parallel leg SU has been increased by the full penetration SS′ and that the throat has benefited by the constant .0442 in., but that the transverse leg has not been increased at all. From Fig. 3 it can be seen that for the 45 deg. fillet the value of the transverse leg is but 20 per cent greater than that of the theoretical throat. Thus if the strength of the actual throat exceeds 120 per cent of the theo-

retical, the transverse leg becomes the critical section. Fig. 12 shows that a 20 per cent increase in strength corresponds to a 5/16 in. weld, therefore for welds of this size and below, the increased strength of the actual weld over the theoretical, will be considered 20 per cent.

Due to the fact that a constant penetration is used with varying sizes of welds, it is necessary when discussing actual welds, to select a definite size.

(2) *The Influence of Shape*

Fig. 13 shows the efficiency of the elements of a 3/8 in. actual fillet weld as the angle α varies from 0 deg. to 90 deg. It will be seen that the

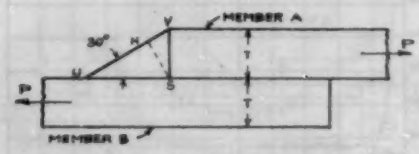

FIG. 14. THEORETICAL FILLET WELD REINFORCED BY LENGTHENING LEG PARALLEL TO FORCE

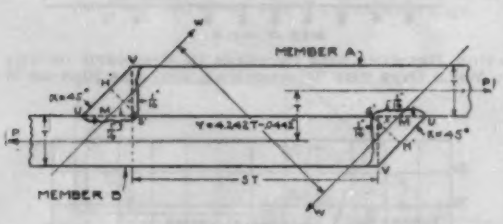

FIG. 15. ACTUAL LAP JOINT IN TENSION

curve for the efficiency of the transverse leg SU is just the same as that shown in Fig. 3, since the value of that leg is not affected by penetration. The curve for the parallel leg has been lifted, due to the increased strength brought about by the penetration, and the curve for the efficiency of the throat has also been raised.

It will be noted that at 45 deg. the efficiencies of throat and the transverse leg are nearly equal. This means that in actual fillet welds the strengths of the throat and the transverse leg are practically balanced. If this be true, then reinforcement by increasing the throat of the weld as shown in Fig. 5, or by lengthening the parallel leg as shown by Fig. 14, will be of little benefit, unless additional bonding can be obtained.

Since fillet welds generally fail through the throat and since by dealing with the throat only simply stresses are involved, the throat will be considered the critical section of fillet welds in tension and all strength tables should be based on the *actual* throat values.

Therefore, the theory bears out what has already been learned by

experience, that in tension, the 45 deg. fillet weld is the simplest, most efficient and most economical form of fillet weld.

(3) *The Fillet Welded Lap Joint*

In the *actual* lap joint shown in Fig. 15 the couple PT remains as before. The forces W now pass through the center of the actual throat S'H', and therefore the distance between them has been reduced by the amount 1/16 in. sin α = .0442 in. As was stated before, in discussing actual welds it is necessary to consider a specific size of weld. If the welds in Fig. 15 were ½ in., the efficiency of the joint would be about 50 per cent. The efficiency of double fillet-welded lap joints in tension, taking into consideration eccentricity, can be readily tabulated for the ordinary range of sizes.

It will be found that when the throats of the welds are stressed to 36,000 lb. per sq. in., that the efficiency of the joint will be somewhat

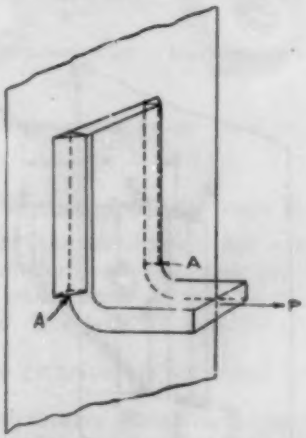

FIG. 16. FILLET WELDS SUBJECTED TO PROGRESSIVE TENSION

greater than that indicated by the calculations. This is due to the fact that the deflection of the joint has reduced the eccentricity.

(4) *The Influence of Not Obtaining Metal-to-Metal Contact*

If, in the actual weld shown in Fig. 11, the surfaces were not in metal-to-metal contact, the forces would pass through the throat and the transverse leg in exactly the same way as they would in Fig. 11, and therefore the strength of the weld would not be affected.

If, in the actual lap joint shown in Fig. 15, the surfaces were not in metal-to-metal contact but were .25 T apart, then the bending couple would be increased to 1.25 PT but the resisting couple would have its lever arm increased by the amount 0.25 T $\times$ 0.707. When the weld

size is ½ in. the efficiency of the joint is about 47.8 per cent. Therefore, it appears that in actual fillet-welded lap joints in tension, even with relatively large distances between the faying surfaces, the drop in efficiency is not great and for all practical purposes the influence of not obtaining metal-to-metal contact between the faying surfaces may be neglected.

C. Progressive Tension

Under the action of the force P as shown in Fig. 16, maximum tensile stresses will occur at point A. The relatively small ductility of the weld metal will permit but little distribution of the load before failure occurs at A. When this takes place the edge has been nicked and a tear started, and under these conditions the concentration of stress is intensified. Therefore, once the tear has started, considerably less force is required to maintain it, and rupture rapidly proceeds to the ends of the welds. At no time is the entire weld under stress, only a small part is resisting the force P, and the stress in this part is not uniformly

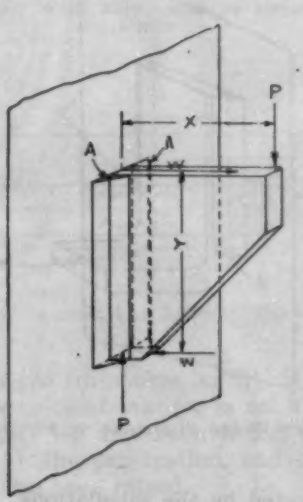

Fig. 17. Fillet Welds Subject to Combined Shear and Progressive Tension

distributed, and the area of this part is not determinate. The action that occurs under the conditions illustrated in Fig. 16 is exactly the same as takes place in tearing an adhesive plaster from the body. Since the stresses in a weld subjected to progressive tension cannot be predicted, any design that gives rise to progressive tension is bad design.

Fig. 17 illustrates another and very common case of progressive tension. The downward force P is balanced by the shear in the welds, but the couple PX must be balanced. PX is equivalent to another couple

WY, and this latter couple causes progressive tension at A. Such design is theoretically wrong.

IV—FILLET WELDS IN COMPRESSION

A. Theoretical Welds

(1) *45-Degree Fillets*

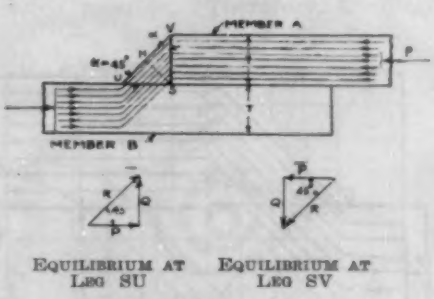

Fig. 18. Theoretical Fillet Weld in Compression

Fig. 18 shows a theoretical full fillet weld in compression.

It will be seen that the conditions for compression are the exact opposite of those in tension, and therefore the throat of a fillet weld in compression is under pure compression, and this stress is exactly twice that in the base metal. The legs SU and SV are under combined shear and compression. The principal or combined stress is likewise $1.618 \frac{P}{T}$

This stress is a compressive stress and acts in a direction R, and, as before, the area upon which it acts must be perpendicular to R. But the bonds SU and SV are not perpendicular to R, and therefore fracture probably would occur along the jagged area shown diagrammatically by Fig. 19.

On facets BC, DE, FG, etc., normal to the stress failure would occur by compression, and on facets AB, CD, EF, etc., failure would occur by shear. If the line of fracture were as indicated its length would be a maximum, and would be 1.41 T.

In compression there is no bond; the full strength of the weld metal would be developed if there were no connection between the base metal and the weld metal, and therefore a mean value to apply to the legs as a whole will depend upon the values of the weld metal in compression and the bond in shear. Also this mean value will depend upon the ratio of the areas in shear to those in compression. For the 45-deg. fillet, the areas are equal, and the mean value is the average between the 36,000 lb. per sq. in. and 25,000 lb. per sq. in. or 30,500 lb. per sq. in.

The combined stress is 1.618 times the stress in the base metal, and when the bond is stressed to the mean design ultimate strength, the maximum force that the bond can transmit will be

$$\frac{1.41\ T \times 30{,}500 \text{ lb. per sq. in.}}{1.618} = 26{,}600 \text{ lb. per sq. in. T,}$$

which gives a maximum efficiency of 44.3 per cent for the bond.

Since the strength of weld metal is considered the same in compression as in tension, and since the forces act in exactly the same way except

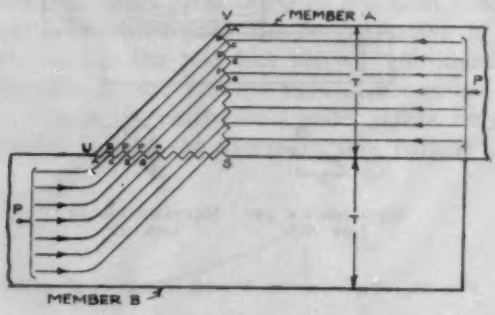

FIG. 19. THEORETICAL FILLET WELD IN COMPRESSION SHOWING HYPOTHETICAL PLANES OF FAILURE IN LEGS

that they are opposite in direction, the design efficiency of the throat of a theoretical full fillet weld in compression is 30 per cent.

It will be seen from the above that the strengths of the elements of the weld are not balanced, that the legs are stronger than the throat.

(2) *The Influence of Shape.*

Consider the conditions when the angle α ranges from 0 deg. to 90 deg. Equations II, III and IV hold for compression as well as tension, but the values to be used with them are different. In Fig. 20 these equations are plotted for compression.

The curve for the throat is identical with that in Fig. 3, because the tension and compression values are the same. It will be noted that the curves for the transverse and parallel legs are much higher than before, and that the former never crosses that of the throat. In other words, at no time is the transverse leg weaker than the throat.

If the face of the weld be made concave as shown in Fig. 4, the stress in the throat will be 3.44 times the stress in the base metal, and the efficiency of the weld becomes 17.4 per cent as before.

If the weld be reinforced by lengthening the throat as shown in Fig. 5, the weld will benefit to the extent that the legs are stronger than the throat or 47.6 per cent. Since the reinforced throat is 41 per cent

longer than the straight throat, it followed that reinforcement of fillet welds in compression is of value.

If the weld be reinforced by lengthening the parallel leg as shown in Fig. 14, the throat will be increased by about 22 per cent but it will remain the critical section.

(3) *The Fillet Welded Lap Joint.*

In compression, the lap joint would not try to line itself up as shown in Fig. 7, instead the eccentricity would increase, and the joint would be useless as a compression member. Therefore, the only occasion that

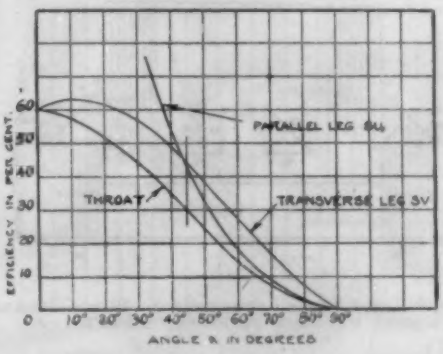

FIG. 20. THE RELATION BETWEEN THE EFFICIENCY OF THE ELEMENTS OF A THEORETICAL FILLET WELD IN COMPRESSION AND THE ANGLE α

such a joint can be used in compression is when it is so stiffened that it cannot move further out of line, and under those circumstances the joint is nothing but two fillet welds and may be figured as such.

(4) *The Influence of Not Obtaining Metal-to-Metal Contact.*

The same reasoning that was used in the case of tension applies to fillet welds in compression when metal-to-metal contact is not obtained between the faying surfaces, and the strength of the weld is not affected.

Since the fillet welded lap joint in compression is merely two individual welds, the absence of metal-to-metal contact between the faying surfaces would have no effect on the strength of the joint.

(5) *Special Case of Compression.*

Often the members are constrained to move in a definite manner, thereby causing failure to occur differently from that which would normally obtain. Such a condition is shown in Figs. 21 and 22.

The conditions of equilibrium at the throats are shown by the vector diagrams. It will be seen that for every horizontal component Q, there

is an equal and opposite component, and therefore these nullify each other, and only the vertical forces P are left. There is no way for the weld to fail but by shearing along the leg SU.

The value in compression of the transverse leg SV is 36,000 lb. per sq. in. T, and its efficiency is 60 per cent. The value in shear of the parallel leg is 26,000 lb. per sq. in. T and its efficiency 43.3 per cent.

Reinforcing by increasing the throat will be of no value because the throat is not the critical section. Reinforcing by lengthening the leg SU, however, will be very efficacious. The value of the leg will become

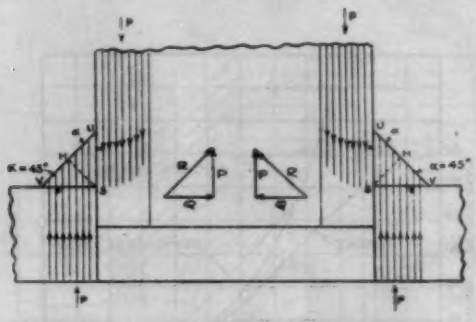

FIG. 21. THEORETICAL FILLET WELD IN COMPRESSION WHEN MEMBER IS CONSTRAINED TO MOVE PARALLEL TO FORCE. CASE I

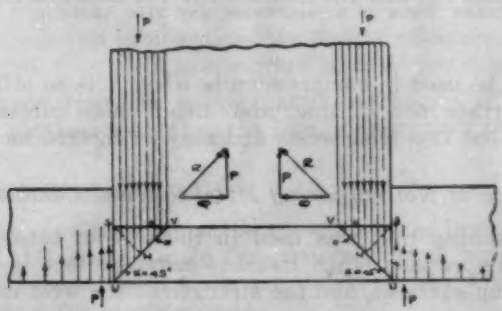

FIG. 22. THEORETICAL FILLET WELD IN COMPRESSION WHEN MEMBER IS CONSTRAINED TO MOVE PARALLEL TO FORCE. CASE II

26,000 lb. per sq. in. $\times$ 1.73T = 45,000 lb. per sq. in. T and its efficiency, 75 per cent. This latter value is more than that of the transverse leg in compression, but it must be remembered that the compressive value of weld metal is greater than the 57,000 lb. per sq. in. chosen, and also, that when a metal is constrained from flattening out, its resistance to compression is greatly increased. Welds on the inside of the bottom plate as shown by figure 22 are considerably constrained, and therefore

efficiencies of welds reinforced by lengthening the parallel leg may under these conditions reach that of the leg SU in shear, or 75 per cent.

B. Actual Welds

(1) *45-Degree Fillets.*

In an actual weld such as shown in Fig. 11, the effect of the penetration is the same as it was in tension, the throat is increased to S'H' and the parallel leg is increased by the amount of the penetration, and the transverse leg is benefited not at all.

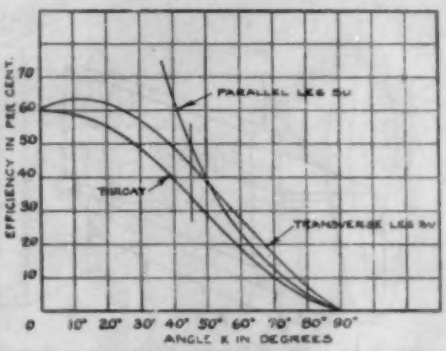

Fig. 23. The Relation Between the Efficiency of the Elements of an Actual ⅜ In. Fillet Weld in Compression and the Angle α

The increase in strength of the actual weld over the theoretical is shown by the curve in Fig. 12. In compression, however, the efficiency of the transverse leg SV is much greater than in tension, as shown by Fig. 20 and therefore an increase in strength of about 50 per cent would be permissible. This corresponds to a weld size of ⅛ in. Hence, the curve in Fig. 12 will be considered straight below that size.

It must be pointed out that it is difficult to make an actual fillet weld in a compression test fail through the throat. The root of the weld is very liable to suffer from bad bonding, and a flaw between the weld and the member B will start a tear. Once such a tear has started, the eccentricity of member A increases and the tear progresses along the parallel leg SU, and the failure of the weld is not by simultaneous compression but mainly by progressive tension.

(2) *The Influence of Shape.*

Fig. 23 shows the efficiency of the elements of an actual ⅜ in. fillet weld as the angle α varies from 0 deg. to 90 deg. It will be seen that the throat and the parallel leg have been lifted, but that the transverse leg is the same as it was for the theoretical weld. However, at no time is the strength of the throat greater than that of the legs.

If the weld be reinforced by lengthening the leg SU, the strength will be increased some 40 per cent, but the throat will remain the critical section.

(3) *The Fillet Welded Lap Joint.*

Since the fillet welded lap joint in compression may be considered two simple welds, the preceding discussion applies to this type of joint.

(4) *The influence of Not Obtaining Metal-to-Metal Contact.*

The strength of the weld or a lap joint would not be affected by the absence of metal-to-metal contact between the faying surfaces.

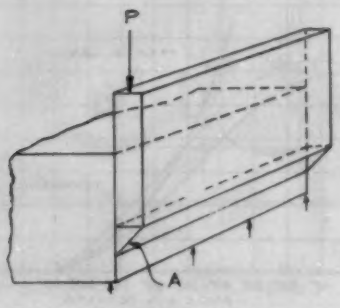

FIG. 24. FILLET WELD IN PROGRESSIVE COMPRESSION

(5) *Special Case of Compression.*

In the special case of compression shown in Figs. 21 and 22, the transverse leg will not be benefited by penetration and the increase in the length of the throat will not add to the strength of the weld because the weld cannot fail through the throat, but the parallel leg SU will gain the full depth of the penetration. It should be pointed out that the bond between the weld metal and the base metal along this leg is not the critical section, failure will occur by shear through the theoretical leg SU.

C. PROGRESSIVE COMPRESSION

The same reasoning applies to fillet welds subjected to progressive compression as shown in Fig. 24, as applied to progressive tension. The stress in such welds is indeterminate and the design that evokes such stress is bad.

V—FILLET WELDS IN TRANSVERSE SHEAR

A. THEORETICAL WELDS

The term transverse shear as used throughout this discussion designates pure shear. At the time this term was selected, a weld normal to the force P, such as is shown in Fig. 36, was believed to be in shear, and

the adjectives transverse or normal were used to distinguish such welds from those in parallel shear. Analysis, however, revealed the fact that welds said to be in normal shear are not in shear at all, they are in tension or compression, they differ in no particular from the welds shown in Figs. 1 and 18.

Further consideration showed that the only way that a weld can be subjected to pure shear is to be deposited on a member in torsion as shown in Fig. 25.

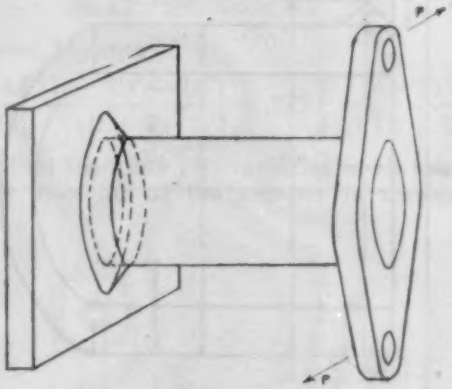

FIG. 25. FILLET WELD ON MEMBER IN TORSION

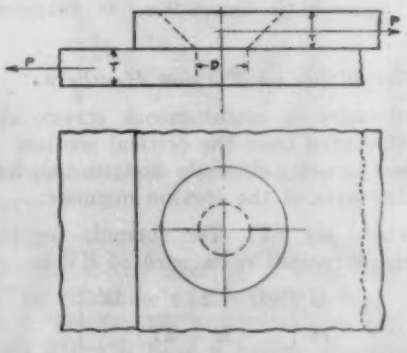

FIG. 26. THEORETICAL PLUG WELD IN TRANSVERSE SHEAR

The term transverse shear, however, is a misnomer for this type of weld, because not only is the weld not transverse to the forces P but the shear in the weld is parallel to the weld; in other words such a weld is truly in parallel shear, and as will be shown in the subsequent section welds said to be in parallel shear are not in pure shear.

In the plug weld shown in Fig. 26, the shear across the diameter D

is contaminated by bending due to the eccentricity, and in Figs. 21 and 22 the shear across the leg SU is likewise not free from bending.

But since the term transverse shear conveys a good impression of what is generally meant by the word shear and since it is necessary to have an adjective to distinguish such shear from parallel shear, it was decided to use the term transverse shear to designate pure shear.

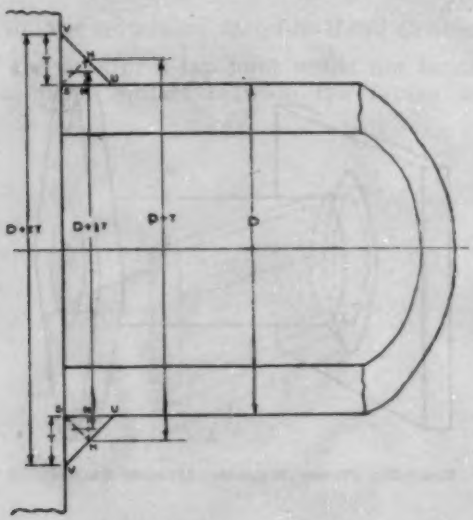

FIG. 27. THEORETICAL FILLET WELD IN TRANSVERSE SHEAR

(1) *45-Degree Fillet Welds on Torsion Members.*

Heretofore in all cases of simultaneous stress, all the stresses have been uniformly distributed over the critical section. In the weld shown in Fig. 27, the stress is not uniformly distributed, but varies directly as the distance from the axis of the torsion member.

Consider the vertical leg SV. The formula for the shearing section, modulus, that is, the torsional resistance of SV is

$$\frac{\Pi}{16}\left[\frac{(D+2T)^4-D^4}{(D+2T)}\right] \qquad V$$

Consider the horizontal leg SU. The resistance to shearing of this leg is its area times the lever $\frac{D}{2}$ or

$$\Pi\, DT \times \frac{D}{2} = \frac{\Pi\, D^2T}{2} = 1.5708\, D^2T \qquad VI$$

Consider the throat SH. The formula for shearing section modulus is based on the assumption that the forces are all acting in a transverse

plane. But the throat in this case is not a plane, it is a truncated cone. The formula for the shearing section modulus of the plane HN between the limits D and (D + T), is

$$\frac{\Pi}{16}\left[\frac{(D+T)^4-D^4}{(D+T)}\right] \qquad \text{VII}$$

Since the conical area SH remains within these limits, the lever arms of the elementary particles are not increased; only the area. Since the area of SH equals $\frac{HN}{\sin 45^\circ} = 1.41\ HN$, the shearing section modulus of the conical throat becomes

$$1.41 \times \frac{\Pi}{16}\left[\frac{(D+T)^4-D^4}{(D+T)}\right] = .277\left[\frac{(D+T)^4-D^4}{(D+T)}\right] \qquad \text{VIII}$$

It will be noted that there are two variables in all these equations, the weld size and the diameter of the member in torsion. In order to

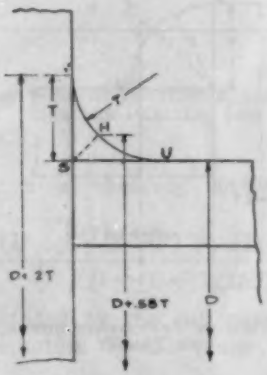

FIG. 28. THEORETICAL CONCAVE FILLET WELD IN TRANSVERSE SHEAR

make clear the relative strengths of the three elements of a theoretical fillet weld in transverse shear, assume a weld size of ½ in., and a diameter of 4 in. The shearing section modulus of the leg SV will be found to be 14.48 in.^{3} The leg SU will have a resistance to shearing of 12.59 in.^{3} The throat SH will have a conical shearing section modulus of 9.48 in.^{3} Therefore, for the size of weld, and diameter of member chosen, the throat is the critical section, and failure will occur at that point.

(2) *The Influence of Shape.*

From Fig. 28 it can be seen that when a weld is concave, not only is the area of the throat reduced from 0.707 T to 0.41 T, but the diameter of the extreme fiber is reduced from (D + T) to (D + 0.58T), and the

diameters enter into the equations as third powers. Using the same sizes as before, T = ½ in. and D = 4 in., and substituting the proper values in equation VIII, the conical shearing section modulus of the throat will be found to be reduced from 9.48 in.³ to 5.3 in.³ Therefore, making such a weld concave reduces the effectiveness greatly.

From Fig. 29 it will be seen that when the weld is reinforced by increasing the throat, not only will the area be increased from 0.707 T to T, but the diameter of the extreme fiber will be increased from (D + T) to (D + 1.41T), and, as was pointed out before, these diameters enter into the equations as third powers. Using the same sizes as before and substituting the proper values in equation VIII it will be found that the conical shearing section modulus of the throat has been increased from 9.48 in.³ to 13.95 in.³ As was shown in the discussion of the 45-deg. fillet, the resistance to shearing of the leg SU is only 12.59 in.³ and therefore, for the size and diameter chosen, the full benefit of reinforcing could not be obtained.

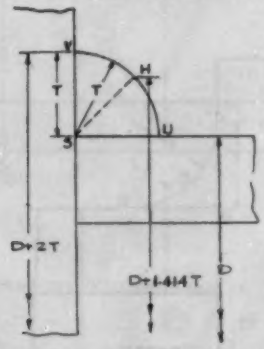

FIG. 29. THEORETICAL FILLET WELD IN TRANSVERSE SHEAR, REINFORCED BY INCREASING THE THROAT

In a weld reinforced by lengthening the parallel leg as shown by Fig. 30 the area of the throat will be increased from 0.707 T to 0.866 T, and the diameter of the extreme fiber will be increased from (D + T) to (D + 1.5T). Using the same sizes as before and substituting the proper values in equation VI and in equation VIII (for 30 deg.) the conical shearing section modulus of the throat becomes 11.95 in.³ and the shearing resistance of the leg SU becomes 21.8 in.³ Therefore in a theoretical weld of the size and diameter chosen, reinforcing by lengthening the parallel leg increases the strength of that leg greatly and makes the throat again the critical section.

B. ACTUAL WELDS

(1) *45-Degree Fillets.*

In the actual weld shown in Fig. 31, it will be seen that the throat has moved in toward the axis of the torsion member and become

S'H'. From the geometry of the diagram it can be demonstrated that the diameters are as indicated in the figure. Substituting these values

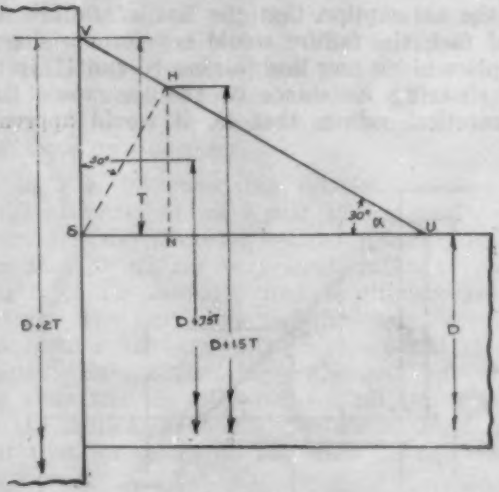

FIG. 30. THEORETICAL FILLET WELD IN TRANSVERSE SHEAR, REINFORCED BY LENGTHENING THE PARALLEL LEG

in equation VIII the conical shearing section modulus of the actual weld becomes

$$0.277\left[\frac{(D+T-1/16'')^4-(D-1/8'')^4}{(D+T-1/16'')}\right] \qquad IX$$

The leg SV has benefitted by the full penetration and, substituting the proper values in equation V, the plane shearing section modulus of S'V becomes

$$\frac{\Pi}{16}\left[\frac{(D+2T)^4-(D-1/8'')^4}{(D+2T)}\right] \qquad X$$

If failure were to occur along the line S'LU, the resistance to shearing would be the resistance of the bond S'L and the plane shearing section modulus of the area LU. The formula for this is

$$\Pi\left[\frac{(D-1/8'')^2T}{2}+1/16\left\{\frac{D'-(D-1/8'')^4}{D}\right\}\right] \qquad XI$$

Assuming the same weld size and diameter of member as before, the conical shearing section modulus of the throat becomes 10.13 in.^{3} The plane shearing section modulus of the vertical leg S'V becomes 15.7 in.^{3} and the resistance of the leg S'LU becomes 13.3 in.^{3}

Therefore in changing from the theoretical to the actual weld, in the case of the weld size and diameter chosen, the section modulus of the vertical leg has been increased from 14.48 in.^{3} to 15.7 in.^{3} The

section modulus of the throat has been increased from 9.48 in.^{3} to 10.13 in.^{3} and the shearing resistance of the leg SU from 12.59 in.^{2} to 13.3 in.^{2} The increase in the case of the last two items was small—about 6 to 7 per cent, and in the case of the parallel leg the formula was based on the assumption that the line of failure would be S'LU. As a matter of fact, the failure would not form a sharp corner at L, it might take place along any line joining S' and U, in which case the value for the shearing resistance of the leg would lie between the actual and theoretical values, that is, it would approach nearer the theoretical.

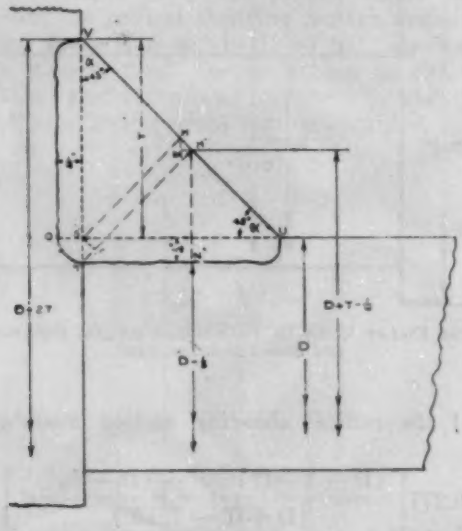

FIG. 31. ACTUAL FILLET WELD IN TRANSVERSE SHEAR

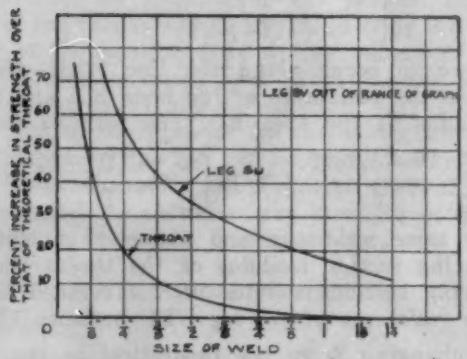

FIG. 32. THE RELATION BETWEEN THE INCREASE IN THE STRENGTH OF THE ELEMENTS OF ACTUAL WELDS OVER THE STRENGTH OF THE THROAT OF THE THEORETICAL WELD, AND THE SIZE OF THE WELD

The curves in Fig. 32 show the increase in the strength of the elements of actual fillet welds over the throat of the theoretical fillet throughout the ordinary range of sizes. The calculations were based on a constant diameter of 4 in. It will be seen that the added strength of the throat drops rapidly with an increase in size and approaches zero in the larger welds. The increased strength of the parallel leg SU is always greater than that of the throat and therefore the throat is always the critical section. The increased strength of the actual leg SV over the theoretical throat is so great that the curve does not show on the sheet.

The curves in Fig. 33 show the relation between the increased strengths of the elements of an actual ½ in. weld over that of the theoretical throat as the diameter of the member in torsion varies. It will be seen that except for very small ratios of diameter to weld size the strength of the actual throat is slightly greater than that of the theoretical. The curve for the throat is flatening out rapidly indicating that further increase in the ratios would result in but little gain in strength. Except for the very small ratios, the throat is always weaker than the leg SU, and for all practical purposes may be considered the critical section. The strength of the vertical leg SV is so great that its values do not show on the sheet.

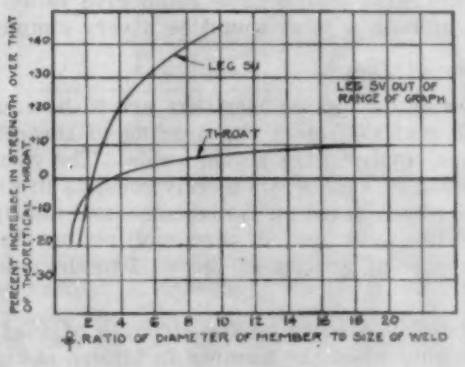

FIG. 33. THE RELATION BETWEEN THE INCREASED STRENGTH OF THE ELEMENTS OF AN ACTUAL ½ IN. FILLET WELD IN TRANSVERSE SHEAR, AND THE DIAMETER OF THE MEMBER IN TORSION

As was pointed out before, all the equations for the strength of welds in transverse shear contain two variables, the size of the weld and the diameter of the member in torsion. Obviously it would be impracticable to make design tables for all sizes of welds and for all diameters. Fortunately this is not necessary. The preceding discussion was intended to show that the strength of the actual welds is but little greater than that of the theoretical for all practical sizes and diameters, and therefore it will be sufficiently accurate and very much simpler to use the theoretical weld in computing the strength of a weld on a torsion member.

Refer to Fig. 27. The resistance of the weld to shearing—what might be termed the approximate shearing section modulus—is the conical area of the throat SH times the distance of its center of gravity to the axis. Expressed mathematically, the approximate shearing section modulus equals

$$[0.707\ T \times \Pi\ (D + \tfrac{1}{2}T)] \times \frac{(D + \tfrac{1}{2}\ T)}{2} \qquad \text{XII}$$

Using the same sizes as before D = 4 in. and T = ½ in. this modulus becomes 10.02 in.^{3} This value is about 5.7 per cent greater than the 9.48 in.^{3} obtained by the correct method. Thus the error due to using an approximate section modulus tends to nullify the error of using the theoretical values, and the net result is sufficiently accurate for all practical purposes.

It will be noted that the first part of equation XII is the depth of the throat multiplied by the number of inches in the length of the center of gravity of the weld, or more simply, the area of the throat times the length of the weld. A design table giving the strength per linear inch of *theoretical* fillet welds in transverse shear would give values which multiplied by the length of the weld would give the shearing strength of the complete weld. The total shearing strength of the weld times the distance to the axis would give the torsional resistance of the joint. This same design table could give values of ½T, so that the computation of such a joint would be a very simple calculation.

(2) *The Influence of Shape.*

Since actual welds on torsion members are to be considered as theoretical welds, the remarks under the heading of theoretical welds other than 45-deg. fillets, apply to the actual welds. The computation for the resistance to torsion of such welds merely consists in finding the proper strength per linear inch based on throat area, and multiplying this value by the length of the weld, and in turn multiplying the product by the distance of the center of gravity of the weld to the axis of the member in torsion.

All of the discussion concerning the strength of fillet welds in transverse shear holds only when the member in torsion is circular in section.

VI—FILLET WELDS IN PARALLEL SHEAR

A. Theoretical Welds

(1) *45-Degree Fillets.*

Refer to Fig. 34, which shows welds in parallel shear. From a consideration of the relative values of the throat and legs as shown in section XX the throat would appear to be the critical section.

Assume, for the moment, that the members A and B are absolutely non-extensible, and that the welds are likewise inelastic. Under these circumstances and *only* under these would the welds fail along the plane of the throat by a simultaneous stress, and that stress would be pure shear.

But all of the materials with which we are concerned are elastic, and the behavior of a joint made of elastic substances is very much different from that described above, and very complicated.

The stress in member A is not at all uniformly distributed. The load is being taken out all along the length of the joint, with the result that the tensile stresses in that member vary from a maximum at end CC to a minimum at end GG. Also since the load is being taken out at the sides, member A is subjected to longitudinal shear varying in intensity from a maximum at the sides to zero at the center. It must be borne in mind that these stresses occur simultaneously, and that the principal stress at any point is a combination of them.

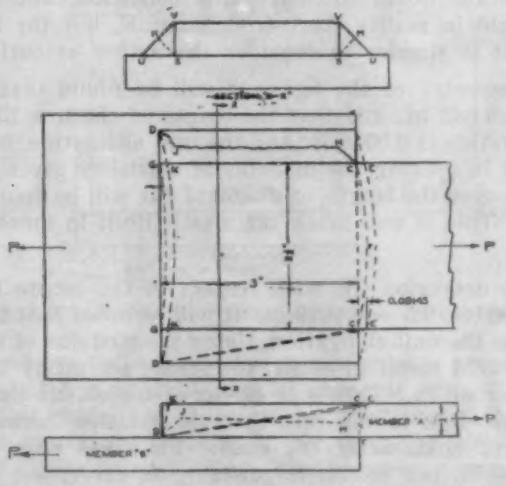

FIG. 34. THEORETICAL FILLET WELDS IN PARALLEL SHEAR

The sides GC elongate by tension, and the points C move out to E. The center of the member A deflects by longitudinal shear, the line EE becomes a curve as shown. The diagonal DC across the face of the weld becomes elongated to DE, and therefore the face of the weld is subjected to tension.

The deflection CE with respect to the weld, that is, to the length CF, is a shearing deflection. Shear deflection is often called *detrusion* and this term will be used to refer to deflection CE with respect to CF.

The end DD deflects and takes up the curved position, DKKD. The curve DKKD is considerably longer than the straight line DD, and this elongation sets up tension forces along the length of the line. Therefore the point D is subjected to tension, not only from the diagonal CD, but from the end of the member A as well.

So it can be seen that a weld, said to be in parallel shear, is, in reality, subjected to combined shear and tension.

In order to convey some quantitative idea of the actions involved let us consider the specific case shown in Fig. 34, and let us deal with simple

stresses only. Assume that the joint is acted on by a force P of 35,000 lb. If the stress in member A were uniformly distributed, the average stress would be 28,000 lb. per sq. in. which is well below the elastic limit of the material. The throat area of the two welds is 2.125 sq. in. If the load were uniformly distributed over this area, the average shear stress would be 16,500 lb. per sq. in. a figure well below the elastic limit of weld metal.

Assume that the stress at point G is zero, and that at point C it is the maximum of 28,000 lb. per sq. in. The mean stress over the length CG will then be 14,000 lb. per sq. in. In a length of 3 in. this stress would produce an elongation of 0.00145 in. It is appreciated that the actual stress distribution would intensify this condition, and also that this elongation would in reality start from point K, but for the purpose of this example it is simpler to consider the action as outlined above.

From the geometry of the figure it will be found that the length of the line CD is 3.082 in., and that the length of the line ED is 3.0836 in. The total elongation is 0.0016 in. and the unit elongation, 0.0016 in./3.082 in. = 0.000519 in. Using the modulus of elasticity given in table I, the uniform stress over the length of diagonal CD will be found to be 15,500 lb. per sq. in. This is well below the elastic limit in tension of the weld metal.

Consider the detrusion CE with respect to the length CF. The unit detrusion is 0.00145/0.5 = 0.0029 in. It will be noted that this unit detrusion is 5.6 times the unit elongation. Using the modulus of elasticity given in table I for weld metal in shear, the stress set up by this strain will be 31,900 lb. per sq. in. This is in excess of the elastic limit of the weld metal in shear. Due to the fact that the relation between stress and strain no longer holds after the elastic limit has been exceeded, this value of 31,900 lb. per sq. in. is probably in error, but whatever may be its absolute value, it is above the elastic limit. Therefore, even though the average value of the stress in the weld is apparently safe, at point C the weld is stressed beyond the elastic limit.

It will be noted from table I that the value of the elastic limit in shear is only about half that of the shearing ultimate strength, that is, the weld metal in shear acts as if it were almost as elastic as base metal. Hence after the yield point has been reached the weld will permit considerable further detrusion without failure, with the result that tension across DE becomes a larger factor. And the greater the deflection of member A the greater the tension along the line DKKD, with the result that point D also becomes a critical point. Failure will start at either one or both of these points depending on how the various stresses add, and once rupture has started it progresses rapidly throughout the weld. Often it occurs along a line such as CHJD. Thus a weld in parallel shear gives way by progressive failure.

The above discussion has not *solved* the problem, it has merely *indicated* it; only simple stresses have been considered whereas all the stresses involved are compound, consisting of tension and shear combined in various ways. General theoretical formulae involving three-dimen-

sional calculations would be far too complicated to be of any practical value, but the above analysis affords a very convvenient means of determining the various factors that affect the strength of welds in parallel shear.

Even though failure may occur along an irregular area such as CHJD, this area could not be considered in design. Except for the ends, this area is the throat of the weld, and therefore the throat will be considered to be the critical section.

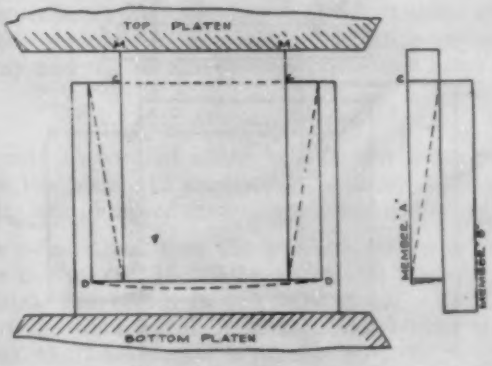

FIG. 35. THEORETICAL FILLET WELDS IN PARALLEL SHEAR. (TESTED BY COMPRESSION)

As was pointed out before, any design that evokes progressive failure is bad design, but the weld in parallel shear is one of our most important structural welds, and therefore we must accept this progressive failure and find an average value in lb. per sq. in. of throat section, that will be safe for all conditions. There are, however, a number of factors that influence this average value for parallel shear.

(2) *The Influence of Size.*

The size of the weld influences the strength of the weld in three ways. The diagonal CD will be longer, permitting a greater elongation. The greater length CF will permit a greater detrusion CE, and the increase in the detrusion will be relatively larger than that of the elongation. The line DKKD will be lengthened and the greater deflection will tend to lessen the intensity of the tensile forces along its length.

(3) *The Influence of Length.*

If the weld is short in comparison with its size, the permissible detrusion will be great relative to the permissible elongation, and the tension along the diagonals CD will be a greater factor. After a certain length has been reached, however, tension ceases to be as important as shear, and further increase in length will have but little effect upon the average value.

(4) *The Influence of Shape.*

If the weld in Fig. 34 be reinforced by lengthening the leg SU, the

diagonal CD will be lengthened, but only in such welds as the diagonal is a determining factor will the shape have much influence on the average value.

(5) *The Influence of Distribution of Stress in the Base Metal.*

If a welded joint in parallel shear be tested as shown in Fig 35, the average stress in the throat of the welds will be appreciably increased.

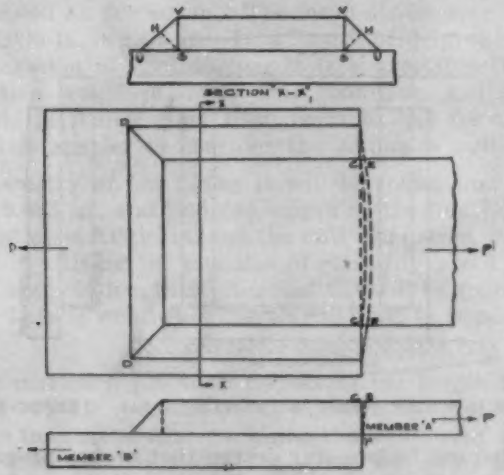

FIG. 36. WELDED JOINT WITH THEORETICAL FILLET WELDS IN PARALLEL SHEAR AND IN TENSION

The test pieces must necessarily be short, and the rigid upper platen of the testing machine holds the top of member A in a straight line, MM and prevents the excessive deflections that occur when welds in parallel shear are tested in tension. The result is, that the member A behaves as if it were more rigid, more like the hypothetical inelastic substance, and the failure is more nearly that of pure shear.

The same thing occurs when a welded joint such as shown in Fig. 36 is tested in tension. The weld at end DD holds that end more or less in alignment and failure of the welds in parallel shear occurs much the same as is the case with those tested in compression. Point C becomes the critical point. In the case of combined side and end welds it is impossible to apportion to the welds in parallel shear, their exact share of the load.

Therefore it can be seen that the distribution of the stress in the base metal has an important influence on the strength of welds in parallel shear.

(6) *The Influence of Intensity of Stress in the Base Metal.*

Since the deflections of the base metal, which is ductile and capable of deflecting, have a great influence on the weld metal which is not so

ductile, the principal factor causing these deflections, namely, the intensity of the stress in the base metal, has a great influence on the average strength of fillet welds in parallel shear. If the stress in the elastic base metal were zero, the metal would behave as if it were inelastic, and there would be no deflection, and the welds would fail by pure shear. Therefore, the lower the stress in the base metal, the more the joint approaches the hypothetical joint, and the closer the average stress would approach that of pure shear.

Since the mathematical computations are so involved, we must have recourse to experiment to determine the influence of these various factors upon the strength of the welds. A large series of tests is already under way and the results of these tests will form the subject of a later paper.

B. Actual Welds

The actual weld in parallel shear will be the same as that shown in Fig. 11. Since the tests will necessarily be made with actual welds, the data and conclusions obtained from these tests will apply to actual welds.

It should be pointed out that the average stress in any member of a structure does not exceed 16,000 to 18,000 lb. per sq. in. and is often much less. Since the stress in the base metal exerts an influence on the strength of the weld metal, it is only just to the welds that experiments in which the base metal is stressed up to and possibly beyond the elastic limit, do not be used as a basis for design data. It is believed that many of the erratic results that have been obtained from tests of welds in parallel shear are due to a variable stress in the base metal.

VII—CONCLUSION

Thus far this paper has been confined wholly to fillet welds. The general theory of stresses in welds embraces all kinds of welds, butt, corner, edge, plug, etc., and bending stresses and all other types of stress. The paper is already very long and it will be necessary to deal with these other points in a subsequent paper.

As was stated in the introduction, it is hoped that the theory, the reasoning advanced herein, will be the means of interesting others in this fertile field for thought, and with many minds at work on the problems, the theory of stresses in welds will soon be established.

Inspection of Welds and Selection of Welders*

C. C. Watson†

BEFORE applying the metallic arc scientifically to steel structures, much experimental work was necessary. A pioneer in the use of arc welding and largest user in the world of arc welding in its own plants, gladly assumed this obligation and the major portion of the results can be obtained.

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†Contract Service Dept., General Electric Co.

The testing of welds in building construction can be classified as follows: Longitudinal sheer on fillets; transverse shear on fillets, involving also simultaneous tensile or compressive stresses on some planes within the fillets, tensile and compressive strength of welds in butt joints. Longitudinal shearing values of fillets are sufficiently well established for safe design of ordinary connections. Also, the tensile and compressive strength of welds in butt joints has been given a great deal of attention. The former are useful when designing joints in pipes and cylindrical tanks, while the latter are of value in planning joints, such as butt splices, in some compression members, for example, top chord splices. Many tests are at present in process which add daily to the already considerable amount of data now available.

Qualifications of Welders

There is a tendency to exaggerate the importance of the human factor in arc welding. It is certainly desirable to have good welding operators, but over-zealousness in this particular is not wise, since it tends to create a skepticism regarding the safety of welds. This increases the cost unnecessarily, because it is likely to burden welders with rules not applied to other workmen, such as riveters and concrete workers, and thus creates unnecessary municipal bureaus.

On important buildings or bridge work the employment of welders merely upon their claims of experience in arc welding is not sufficient. While their claims may be perfectly correct and made in good faith, nevertheless it is true that a welder may be capable of obtaining excellent results in one class of work and yet be unable to perform well the particular work at hand. Each man should be tested by at least one of the following methods:

Butt Welds

Welders should be required to weld four sample butt joints, each made of two pieces ½ in. x 9 in. x 12 in. to form a piece about 12 in. x 18 in. prepared with a single "V." It is recommended that the spacing of these plates be left to the operator making the test, and that the weld be made in two passes. Two of these sample plates should be welded in a flat position and two in a vertical position with the joint located vertically. Each sample plate should be machined to reduce the joints to the thickness of the base metal. From each sample plate the standard 2 in. tensile test specimens should be made and tested in tension. The average tensile strength of each group of two sample plates should not be less than 45,000 lb. per sq. in., and the tensile strength of the lowest in each group should not be less than 40,000 lb. per sq. in. Steel for the "V" joint should be beveled not less than 30 degrees on each piece to form an open angle of not less than 60 degrees.

Lap Welds

Welder should be required to weld four sample lap joints each made of ½ in. x 6 in. x 8 in. plates, clamped one on the other with the 8 in. edges lined up, but the 6 in. edges offset ½ in. A full ½ in. fillet weld should then be made along one 6 in. edge. Upon completion of the weld

and after cooling, the specimens should be torn apart and broken by wedging at the unwelded 6 in. edges and the fractured metal of the specimens broken through the weld to show metal—bright, dense, even textured, either crystalline or fibrous, irregularly torn, and void of iridescent colors, good fusion of weld and base metal, and good penetration into the right angle corner of the fillet. All lap joints of ½ in. to 11/16 in. steel should be fillet-welded in two layers. The contour of the cross-section of the fillets should be as near as is practical to a triangle with equal sides and with minimum size not less than the prescribed dimensions of cross-section of fillets.

Fillet Welds

Welders should be required to make three fillet weld sample test specimens as follows—each specimen to consist of two main plates 4 in. x 1 in. x 12 in. placed in line with their inner ends, separated ½ in., making the over-all length 24½ in. These plates are to be connected by two splice plates, each 3 in. x ¾ in. x 6½ in., symmetrically clamped to the main plates, one above and the other below the two main plates, and each welded in a flat position to each main plate by 2½ in. x ⅜ in. triangular fillets with their craters filled, each of these fillets to be started at outer ends of the 3 in. x ¾ in. x 6½ in. spliced plate and continued 2½ in. along the 6½ in. dimension toward the center, thus leaving a 1½ in. space between the inner ends of the fillets. The specimen should then be inverted and the four remaining fillets also deposited in the flat position. When depositing the weld metal, craters should be filled to full cross-section of the fillet. Electrodes of 3/16 in. diameter should be used. These specimens are to be tested in tension to ascertain the longitudinal shearing value of the ⅜ in. fillet welds and the fractured specimens should be examined for: Penetration into, and fusion with, base metals, freedom from gas holes and from slag inclusions in fillets. The three specimens should show at least an average ultimate longitudinal stress of 42,000 lb. per sq. in. and the lowest in the group should show at least 38,000 lb. per sq. in., of minimum section of fillet.

Of these test methods, that to determine the longitudinal shear value, this one is the best, but the most expensive one, for ordinary building construction involving trusses, beams, girders and columns, the test specimens just spoken of is designed in such a manner that, when the 10 in. of fillet reach their ultimate shearing strength, the unit tension in main and spliced plates will not materially exceed the elastic limit. In fairness to operators, they should be given an opportunity, before making test specimens, of familiarizing themselves with the type of electrode and current density to be employed. It is also desirable to test welders in this manner from time to time during the progress of the work.

The authorities of one city have recently suggested that all welders employed be licensed by the municipal bureau of building inspection. Such action seems unnecessary. However, it is necessary that the engineer and architect in charge of a welded steel building frame have in their employ a competent inspector, who qualifies the welders by any or all of the methods just described. If municipal regulation of any kind is necessary, it would seem better to have the contractor for the steel sub-

mit evidence that the welders he proposes to employ on the building in question have met these or similar qualification tests.

Inspection

As in riveted steel and in reinforced concrete construction, the conservative engineer will provide for inspection of all welded work both in the shop and in the field. It is the duty of the inspector to see that the sizes and lengths of steel sections are correct, that welders are well qualified, that the proper current and electrodes are used, that the current values are changed to suit the various sizes that may be used. That the sizes and lengths of fillets are deposited to conform with those specified on the drawings, as to size and position.

A good inspector can learn much by visual inspection of a fillet weld, or any weld, for that matter. This has been proved by tests which a large company in the East has made. Rounded edges denote lack of penetration of the weld into the parent metal, or into the parts being welded, one short and one long side of a triangular fillet indicates that the electrode wire has been held at an incorrect angle while welding. A crater at any point in the fillet, other than the end, is evidence that the arc has been broken and the fillet not continuously laid. Numerous gas holes on the surface of the fillet or weld indicate too long an arc and lack of penetration. A current shown by the ammeter on the welding machine to be too great for thin plates, or too light for thick ones, is undesirable and the inspector should see that the welders adjust their machines to obtain the current suitable for the thickness of the material being welded.

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